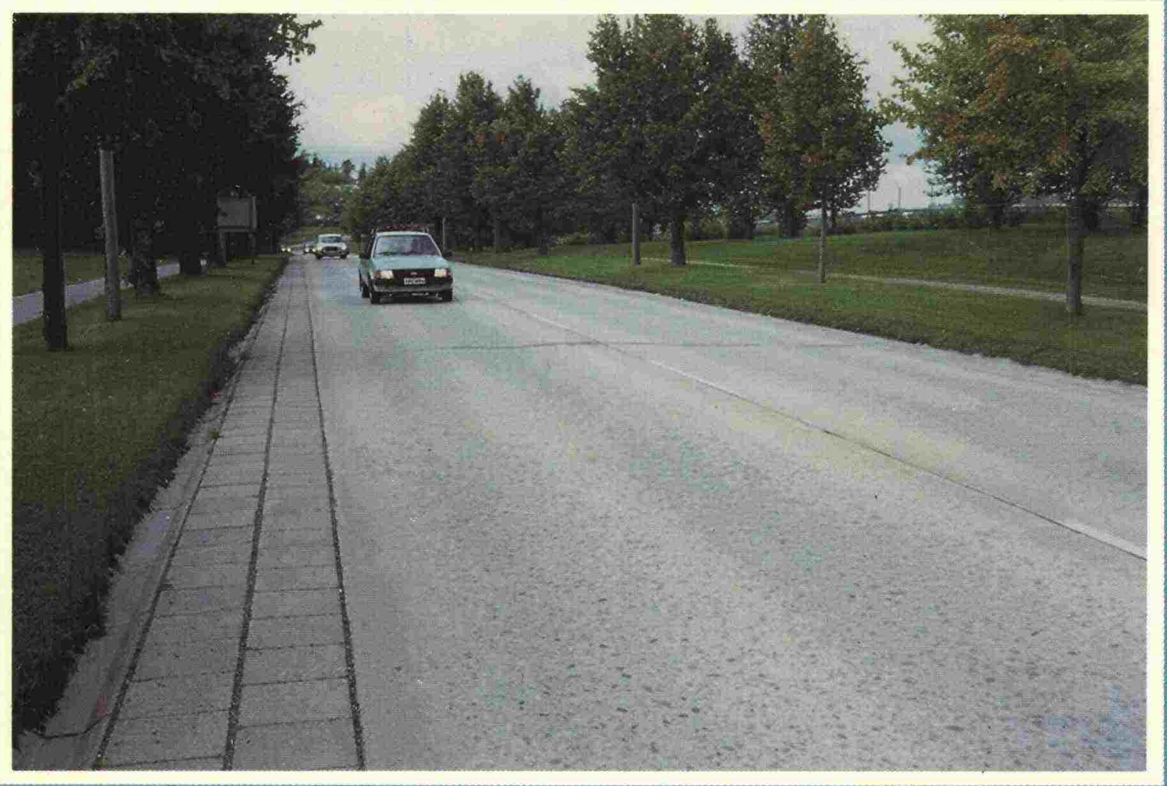
CEMENT-TREATED AND CONCRETE PAVEMENTS:

their applicability in Finland and elsewhere



THE FINNISH ASSOCIATION OF
BUILDING MATERIALS INDUSTRY

ROADS AND WATERWAYS ADMINISTRATION

TVH 723869 / 1989

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AND
CONCRETE PAVEMENTS:

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PREFACE

Roads are today exposed to increasing load and abrasion all over the world. The heavy traffic is still growing, the roads are aging and beginning to deteriorate. The society is not willing or capable enough to finance investments for the improvement of the road network. New knowledge of the durability of constructions and new technically and economically profitable solutions are now needed. One such solution, which has been known before but which now again arouses special interest, is the use of cement as a binder in pavement layers and as a concrete pavement.

The advantages of cement are known in principle also in Finland, but our experiences are restricted and we have not yet enough knowledge of applicability of cement in our cold climate. For this reason a research project was established in autumn 1986 in co-operation between the cement industry and the Roads and Waterways Administration to collect the latest available information of the applicability of cement treated and cement concrete pavements in the Finnish conditions. Especially the following points were chosen for the investigation:

- use of cement-treated pavements when improving the bearing capacity of a road structure
- abrasion resistance of concrete pavements compared with asphalt pavements in a studded tyre traffic
- rehabilitation of concrete pavements
- behaviour of concrete pavements on a weak and compressible soil
- design of concrete pavements in frost action conditions

The Research was done by Mr Jussi Rahiala (M.Sc.) from the Finnish Association of Building Materials Industry (Rakennusaineteollisuusyhdistys). A working team formed by experts from RWA and cement industry has supported him in his work. The management team, where I was chairman, consisted of the following members: Aulis Miettunen/Oy Partek Ab, Jan Owren/Oy Lohja Ab, Jyrki Malmio/Rakennusaineteollisuusyhdistys and Veikko Hakola/the Roads and Waterways Administration. Mr Aulis Nironen, from the Roads and Waterways Administration, acted as secretary of the management team and as chairman of the expert group.

This publication is the report of this project and Mr Rahiala is responsible for its contents. The commissioning parties will use the information and conclusions of this report in training and research as well as in making decisions concerning the use of cement in road structures. We hope that the report will contribute to distributing new information on the possibilities of cement within our professional circles.

On my behalf I wish to thank everyone for their contribution.

Helsinki, September 30, 1988

Väinö Suonio
Roads and Waterways Administration
Director-in-Chief

The English version is made under the guidance of the author and published in November 1989.

ACKNOWLEDGEMENTS

This research on cement-treated and concrete pavements has been carried out mainly as a literary research by studying the Finnish and foreign literature of this field published in the 1970's and 1980's. I have also studied the subject in Scandinavia, Central Europe and North America by interviewing many experts there. Furthermore, I have acquired complementary knowledge by corresponding with many foreign and Finnish experts and interviewing experts in Finland. The Road and Traffic Laboratory of VTT (the Finnish Government Technical Research Center) has also made for my project some track tests and other investigations about wear resistance of concrete pavements.

As a result of the project this report is published with all the up-to-date knowledge and experience I received together with basic data about each subject dealt with. In addition to this report there are many intermediate and travel reports with photographs and an 'article bank' consisting of about 500 volumes of publications and articles used in this work. All this material is stored in the library of the Roads and Waterways Administration. A separate study of wear resistance of corirete pavements has been published as research No 658/1988 in the Road and Traffic Laboratory in VTT.

This project has been a 'one-man-project', but the support and help which I have received from many persons in acquiring information, in handling the material and in completing the report have been most valuable.

The following persons have belonged to the expert group of the project:

Aulis Nironen, Road and Waterways Administration, chairman
Reijo Orama, Road and Waterways Administration
Pauli Haapakoski, Oy Partek Ab
Martti Hintikka, Oy Lohja Ab (from 18.8.1987 on)
Arto Rosama, Oy Lohja Ab (up till 18.8.1987)
Jussi Rahiala, RTY, secretary
Anssi Lampinen, VTT road and traffic laboratory, special expert

The expert group has gathered regularly each month. The team has supported the project and guided its contents in an important way.

In December 1987 a distinguished group of Finnish experts in the field of road structure and the use of cement gathered to a seminar, and they gave me valuable information and encouragement.

Data acquisition has mainly been accomplished with the valuable help and expertise of the personnels in the libraries and information services of Oy Partek Ab, Oy Lohja Ab and the Road and Waterways Administration.

Mrs. Heidi Sid and Mrs. Seija Tiainen from Oy Partek Ab have competently handled the foreign correspondence and planned the research journeys. They have been in charge of all the secretarial work of the project as well as of the organization of the article bank.

The report was first published in Finnish in 1988. This English edition is made by Oy Viatek Ltd for the Finnish Association of Building Materials. The translation is made by Mrs. Pirkko Ojala and the completion of the report by Mrs. Raila Laitala. The report was printed by Oy Grafiart Ltd in Turku.

On my research journeys during these two years, I have had the opportunity to meet numerous foreign specialists with whom the discussions have been most interesting. The following persons have particularly contributed to my work:

Sweden:	Mr. Sten Pettersson	Vägverket
	Mr. Örjan Pettersson	VTI
	Mr. Ronny Andersson	Cementa
Norway:	Mr. Torbjoern Naimak	Vegdirektoratet
	Mr. Arne Ramsvik	Vegdirektoratet
	Mr. Nils Pedersen	Norcem AG
	Mr. Torger Baerland	Norcem AG
Denmark:	Mr. Christian F Justesen	Aalborg Portland Cement
	Mr. Knud Puckman	Vejdirektoratet
Germany:	Ministerialrat E Neussner	Bundesmin. für Verkehr
	Mr. Jörg Bermel	Heilit & Wörner AG
	Prof. Josef Eisenmann	Techn. Univ., München
Austria:	Ministerialrat W Jaderny	Bundesministerium für Bauten und Technik
	Dr. Hermann Sommer	Zementforschungsinstitut
Switzerland:	Mr. Kurt Suter	Bundesamt für Strassenbau
	Mr. Willy Wilk	Betonstrassen AG
France:	Mr. Francois Verhee	Setra
Spain:	Prof. Carlos Craemer	University of Madrid
Belgium:	Mr. Guido van Heystraeten	Belgian Road Research Center
	Mr. Frederic Fucs	" " " "
Greece:	Prof. Ap. Yotis	National Techn. Univ.
	Mr. N Marsellos	Public Works Research Center
U.S.A.:	Mr. Andrew D Halverson	Minnesota DoT
	Mr. William A Yrjanson	ACPA, Chicago
	Mr. Robert G Packard	PCA, Chicago
	Mr. Henry M Yamanata	Illinois DoT
	Mr. Richard L Berg	CRREL, Hanover NH
	Mr. David W Bernard	New York DoT
Canada:	Mr. Thomas J Kazmierowski	Ontario MTC

In addition I have received written reports from:

Great Britain:	Mr. B J Walker	BCA
U.S.A.:	Mr. K H Dunn	Wisconsin DoT
	Mr. Thomas A Coleman	Michigan DoT
Canada:	Mr. Daniel Venzina	Ministre des Transport, Quebec
	Mr. J Hosang	Highways & Transport., Manitoba

All of you and many of your colleagues have splendidly assisted my state-of-the-art project and helped me to summarize the latest knowledge about concrete pavements and cement-treated materials. I want to express my sincere gratitude to all of you.

ENGLISH SUMMARY

In Finland cement is quite scarcely used in road construction. Every now and then some experimental concrete pavements and soil cement works have been constructed but the conclusions have remained controversial. Not only the cost-effectiveness but also some technical aspects concerning our severe climate and soil conditions have resulted in hestitation in attitude and practice. But lack of money, lack of good aggregate, increasing amounts of truck traffic and poor performance of road network are also in Finland the reasons for a glaring need of more capable pavement structures. Year by year the interest in cement and concrete in pavements is getting more obvious.

In 1986 a research project was established in order to collect the up-to-date knowledge and experience about cement-treated and cement concrete pavements especially in countries with seasonal frost conditions. This publication is the report of that project. Even though some basic matters and details are treated the report tends not to be a manual or textbook, but a state-of-the art just for the Finnish situation. The report is divided into independent parts with references, each of them dealing with a special subject.

Chapter A - Cement-treated pavements - gives a look to foreign and Finnish design and construction practice of cement-treated pavement layers. The practice is found to vary a lot from country to country. The use of cement-treated pavements is affected also by tradition. A new interest is now obvious also in countries which never have used cement in pavements. The performance of cement-treated pavements is found pretty good even though the tendency to cracks reflecting through asphalt is a problem. Anyhow, efforts are going on to minimize the thickness of asphalt layers and thus to improve cost-effectiveness of the structure. In the report cement-treated pavements are considered to be suitable in Finland on the following terms:

- good drainage
- sufficient frost protection and transition arrangement
- sufficient bearing capacity of the subbase
- sufficient thickness of asphalt
- sufficient frost resistance of cement material

Chpater B 1 - General survey of concrete pavements - tells just briefly about the historical standpoints and the progress which has occurred during recent years on concrete pavemet design and construction techniques. An unreinforced, short slab with vertical transverse joints with dowels is introduced as the most suitable design for Finland.

A brief history of the use of concrete pavements in some European and American countries is also presented in Chapter B 1.

Chapter B 2 - Repair and rehabilitation of concrete pavements- presents the new repair and rehabilitation practice which is found in newly published manuals in several countries. Both faults, causes and repair methods are presented. Special attention is paid to rehabilitation methods for the ruts caused by studded tyres. Up to now the best method seems to be diamond grinding of the rutted pavement. The practice is widely used in USA; the adaption to rehabilitation of ruts is not very popular today. The Norwegians have made some experimental works by cold milling - and have succeeded.

Chapter B 3 - Wear of concrete pavement caused by studded tyres - examines the factors affecting the wear resistance of a pavement. It is found that the most important factor is the wear resistance of the stone material. It is also proved that the compressive strength of concrete is a good indicator; compressive strengths up to 70 MN/m^2 should be favoured. With good aggregate and high compressive strength a wear resistance 2,5...3,5 times that of asphalt concrete (with the same aggregate) can be achieved.

Chapter B 4 - Concrete pavements in severe climate - deals with basic matters like frost action phenomena and frost index measurements quite thoroughly. The main contents is the state-of-the art of the frost protection practice of roads in several countries. It is found that many countries allow shallower protection for concrete than for asphalt pavement because the loss of bearing capacity in spring time is of minor importance concerning concrete pavements. Because of the most severe frost conditions that kind of reduction in pavement thickness is not suggested in Finland.

Chapter B 5 - Concrete pavements on weak and compressible soils- explains the main principles of soil consolidation caused by road embankment and traffic. The most usual methods for reducing the amount of differential settlements are also introduced. The Swiss practice and experience in dealing with concrete pavements on compressible soils is put forward and found most interesting. The present design practice in Finland is stated as follows:

- total settlement during pavement life 250 mm
- longitudinal gradient difference not more than 0,5 per cent during pavement life,
- crossfall difference not more than 1,0 per cent during pavement life,
- settlement increase not more than 30 mm/year in the beginning of pavement life
- safety factor against slide 1.7...1.8.

CHAPTER A
CEMENT-TREATED PAVEMENTS

CHAPTER A

CEMENT-TREATED PAVEMENTS

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CHAPTER A
CEMENT-TREATED PAVEMENTS

A 0
INTRODUCTION

Today road engineers have to tackle many acute problems. It is increasingly difficult to find traditional aggregates of good quality because of either depletion of sand and gravel areas or of enviromental protection. Recycling of the materials which are removed from old pavements or excavations is increasingly demanded for environmental and economical reasons. Experiences in deterioration of old pavements emphasize that structural durability of new roads should be improved. Effective rehabilitation of deteriorated roads calls for new working methods and new ways of thinking as to the quality and use of material. The constantly increasing heavy traffic and the pressure for heavier axle and vehicle loads set ever greater demands on the design of new roads.

These challenges are international. In all developed countries, where economical pressures are ever-increasing, efforts are being made to build more and more durable roads. This calls for research work. And in this work also the old methods will be digged out. Stabilization of soil by using binders - especially cement-treated materials- is again of great interest to researchers.

Also in Finland it can often be seen that the use of mainly unbound structural courses in circumstances where heavy traffic is growing does not guarantee the durability of the structure. Furthermore, from the technical and economical point of view this is not the only structural alternative. This applies especially to base and subbase courses of main roads and main streets and also to the improvement of the bearing capacity of inferior roads. Cement-treated pavements - an old but not much used structural method, which we know from norms and instructions - arouses new interest today.

The aim of this state-of-the-art report based on available investigations and planning instructions is to visualize the possibilities of cement-treated pavements when improving the bearing capacity of a road structure and to find ways for their technical and economical use.

A 1
CEMENT IN SOIL STABILIZATION

The use of cement stabilization in road and street construction is almost as old as cement itself. The first stabilization experiments date back to 1915...1917 to Florida, South-Carolina and England. The method spread first in the United States, and in the 1930s cement-treated and concrete pavements were making strong progress in Germany. During the War the airfields were generally constructed using cement-treated materials.

A general breakthrough of cement-treated materials in road construction took place in the extensive post-war road construction projects all over the world. In the 1950s and 1960s in the United States 40 million m^2 of cement-treated pavement were built annually when the Interstate Highway system was under construction, /1, 2/.

Later the use of soil-cement is stabilized but at the same time it is being used in an increasing number of countries. First the use of cement-treated materials was based on empiricism but in

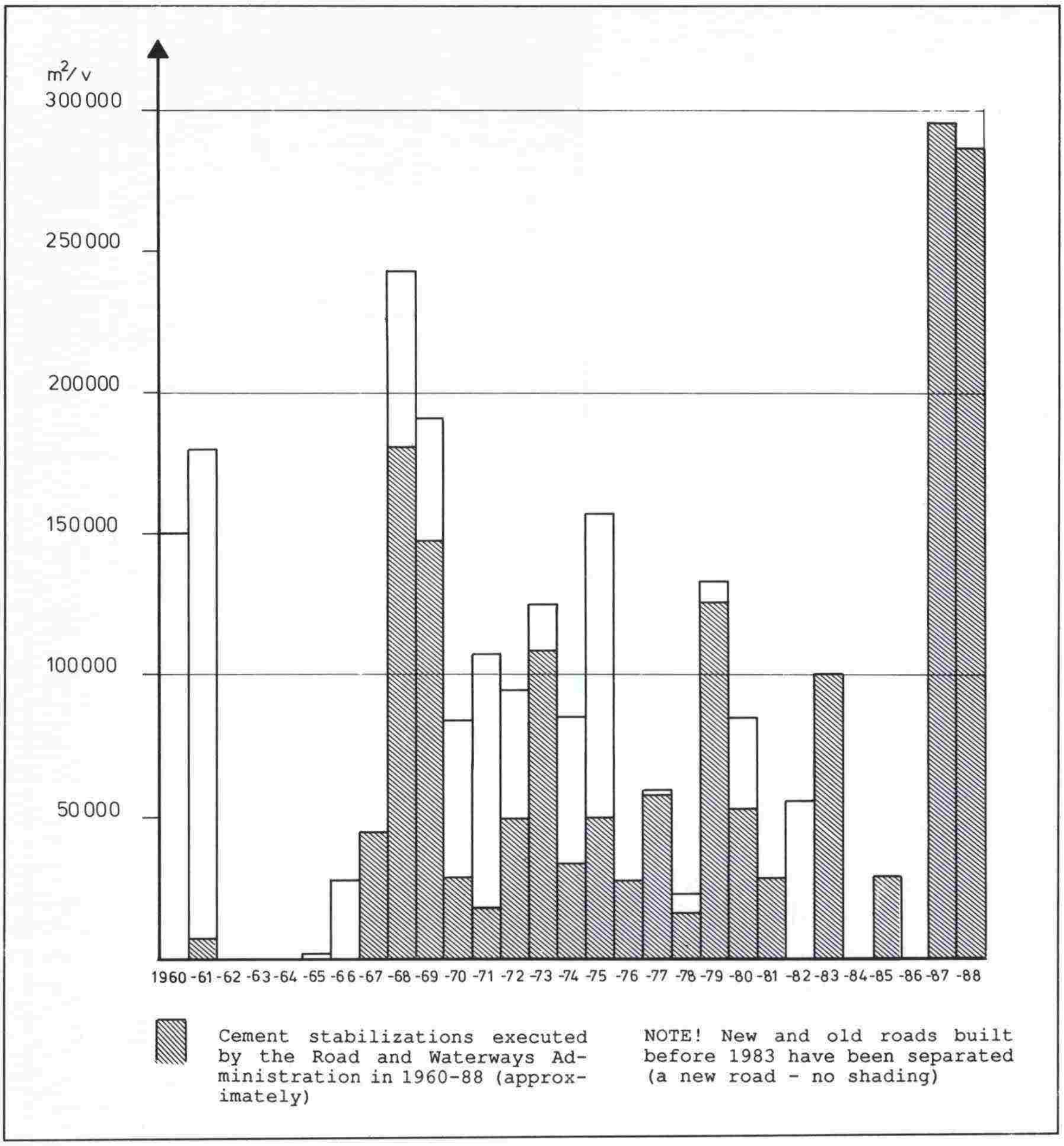


FIGURE A-1. Cement stabilization works on public roads in Finland /37, tvh/

the 1970s or in the 1980s most countries have established - on the basis of their own experiences - their own norms and specifications for a proper design and construction of cement-treated pavements.

In Finland the first cement-treated pavements on a new road were laid in 1960 on the Main Road 6 to Taavetti. In Finland cement-treated materials have been used only on a small scale, approximately 0,1 million m^2 a year (Figure A-1), although the cement-stabilized alternative was already mentioned in the standard instructions by the Road and Waterways Administration as early as in 1964 and although thorough planning and working instructions were available in the beginning of the 1970s /3,4/.

Cement is by no means the only or the oldest binder used for soil stabilization. The use of lime has been known for centuries and it is still used in soil construction to stabilize the types of soil rich in fines. Cut-back bitumens and bitumen emulsions are used in pavements and also in stabilization. The industrial by-products, flyash and blastfurnace slag, are nowadays important additives of stabilization. The binders may compete for the same project, but mostly the correct binder is decided on the basis of the aggregate to be stabilized and of the aims set for the stabilization. Also the availability of a binder is a decisive factor.

As most binders also cement decreases the effect caused by moisture and temperature variations on the soil:

- resistance to water erosion will increase
- resistance to thaw will increase
- frost susceptibility will decrease or is prevented
- processing and compaction properties will improve

The most characteristic feature, however, which addition of cement accomplishes, is the constant increase in strength. The addition of only 1 - 2 % of cement has many affirmative effects: it improves permanently the suitability of frost-susceptible moraine, grading becomes rougher, frost susceptibility decreases or is prevented and the bearing capacity improves. When more cement is added, 3 - 6 %, it makes pure non-cohesive soil or moraine into a stabile layer, which has a good bearing capacity and which can be used as a part of any structural course. Using still greater amounts (>8 %) of cement produces lean-concrete or actual pavement concrete.

The strength is based on the fact that the water in the soil together with cement forms a cement paste, which, when hardening, binds the grains of the soil together. The more bindings between the grains the greater the strength. The compacter the mix the more bindings there are depending on the amount of cement paste. The voids content of cement stabilization is 10-15 %. This value is about ten times higher than that of a normal concrete. This results in the cement-treated pavements being semi-rigid, something between elastic and rigid.

The cement-bound materials are known for their good fatigue resistance, high E-modulus and for the fact that to a certain limit their qualifications are independent of temperature and load. These are very desirable properties in pavements. However, they can be wholly and reliably utilized only by careful preliminary investigations, correct design and by expert construction.

A 2
CEMENT-TREATED MATERIALS AS A CONCEPT

The terms 'cement stabilization' or 'cement-treated pavements' generally refer to binding soils with cement. Cement-treated pavements are considered a high-class stabilization. However, the contents and use of these terms in different contexts is inconsistent and unestablished. In city street construction the term 'lean concrete' is used of the same material, which again has been called a cement-treated pavement. Also internationally the use of these different terms has been inconsistent and this fact has hampered the exchange of information and led to misunderstandings. In the XVIII World Road Congress, PIARC, /5/in Brussels in 1987 a suggestion was made to unify the terms. The use of <u>cement stabilization</u> is restricted solely to subgrades and to the lower parts of a pavement. <u>Cement-treated materials</u> are used in base or subbase courses and they are so designed that the addition of cement permanently increases the strength of the structure. The cement-bound structures which are paved with

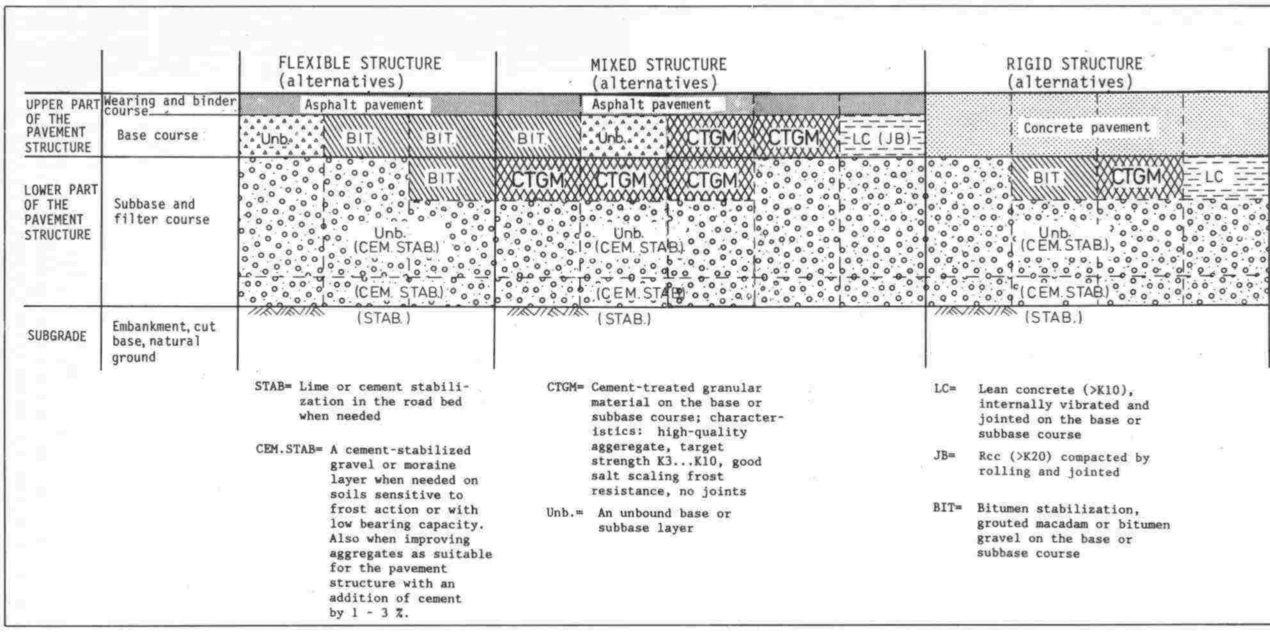


FIGURE A-2. Cement-treated materials and cement stabilization in pavement structures- a principal drawing

asphalt are called either semirigid, composite or mixed structures.

Figure A-2 gives a general view of the names of cement-bound layers and structures. It should be taken as a suggestion for contents of the concepts in Finland.

According to the suggestion the structural properties of cement-treated layers are:

- they are used in base or subbase courses (a layer can be partly or wholly treated with cement)
- they are so designed and constructed that their strength properties can reliably be utilized
- they consist of homogeneous, non-humus aggregate
- they crack freely (no joints)

Later this report will concentrate on dealing mainly with the cement-treated pavements defined here and with their behaviour in the road structure.

A 3
THE STRUCTURAL PROPERTIES OF CEMENT-TREATED MATERIALS

<u>COMPRESSIVE STRENGTH</u>

In a road structure the cement-treated pavements function with the strength which the addition of cement gives to the aggregate. This strength is stated as a compressive strength at the age of 7 days. The target strength can vary 3 - 10 MN/m^2 depending on the thickness of a layer and on its position in the structure. The strengths under the variation range cannot be utilized for dimensioning and so they can be classified as cement stabilization. The strengths exceeding the variation range belong to lean concrete dimensioned in slabs.

The compressive strength of cement-treated pavements is naturally the function of a cement content. With a certain cement content, however, the compressive strength varies greatly depending on the grading of aggregates, dry density of the material, water content, compaction efficiency and curing circumstances. That is why the amount of cement needed for the target strength can be found out only by preliminary tests made beforehand. The accomplished compressive strength can be achieved only by a professional and careful working progress.

In practice the amount of cement varies 4 - 8 % of the dry weight of an aggregate. The lower level of 4 % secures that a cement-treated pavement is frost resistant. Oversized amounts of cement are not only expensive but they also lead to increasing shrinkage cracks and that is why the improvement of the grading of aggregates is more profitable than the use of oversized amounts of cement.

FLEXURAL STRENGTH

As cement-treated materials are used in the upper parts of a pavement also the flexural strength is a vital feature. It is defined by bending beams made as a preliminary test or sawn from a completed pavement. The flexural strength can also be defined from the Proctor cylinders as a so called splitting strength. In different countries the target strengths vary 0,5 - 2,0 MN/m^2, /2/. In Finland we have no definition for a flexural target strength. It can roughly be said that the flexural strength is 1/5...1/8 of the compressive strength. By increasing the compressive strength also the flexural strength will increase. Important factors are the dense graded curvature of aggregates, sufficient amount of rough materials, homogeneity of the mix and an effective compaction in the optimum moisture conditions.

CRACKING

Cement-treated materials are, except stabile and rigid, also freely cracking. Cracking is an inherent feature of cement-treated pavement layers and it cannot and must not be prevented. But if a cement-treated pavement cracks unpredictably it may damage other structures, weaken the serviceability of road surface or at least cause disappointment in the expectations of cement-treated pavements. Thus it is important to know this cracking phenomenon and recognize that it can be influenced with design and construction.

In cement-treated layers cracking occurs for the three following reasons:

a) as shrinkage cracking generally in connection with hardening
b) due to tensions caused by temperature variations
c) due to bending tensile stress caused by dynamic traffic loads

The tensions caused by temperature variations and traffic loads are reduced considerably when the distance of cement-treated material from road surface increases. In a cement-treated base course (a thin asphalt pavement) these tensions are noticeable. They are usually handled by increasing the flexural strength of a cement-treated layer. Shrinkage cracking can be controlled with the following measures:

- no oversized amount of cement
- a long setting time (adding of slag)
- the water content of the mix in the optimum moisture conditions
- a rough, moistened base
- a bitumen film immediately after rolling

The most advantageous cracking system is an integral, quite dense net of cracks. Then the cracks remain narrow micro-cracks which don't endanger the load-transfer capacity, the effect of water or the durability of a structure in general. However, the attitudes towards cracking phenomenon seems to influence greatly on the way cement-treated materials are used in different countries. In some countries reflected cracking through asphalt up to the pavement surface is considered insignificant, a cosmetic fault, and even maintenance because of cracks is not regarded as necessary. In other countries, for fear of maintenance due to cracks, thick (10 - 20 cm) asphalt layers are required on cement-treated layers, /5/.

A 4
DESIGN AND CONSTRUCTION OF CEMENT-TREATED PAVEMENTS

A 41 DESIGN

A 411 Behaviour of cement-treated materials in the pavement structure

Although cement-treated materials have been used for decades to improve the bearing capacity of pavements, the views of their behaviour differ in various countries. Some consider these materials micro-cracking and semi-rigid and think that this mircro-cracking phenomenon should be secured by rolling after the hardening phase has started. Some regard the cement-treated materials rigid and require that the layers should have a sufficient flexural strength in recurrent dynamic loads.

As to design, these different ways of approach lead to different results, /5/:

a) When the design is made on the basis of the flexural strength, the layer thickness must be such that the tensile stresses caused by traffic loads in the lower surface of the layer is 0,5...0,65 times the flexural strength of the hardened mix. This design method calls for aggregate of high quality and for a very careful performance of the work. It is suitable for cement-treated materials in a base course when traffic is very heavy. This method, when completely carried out, leads to rather thick cement-treated layers, even to full-depth structures.

If a cement-treated pavement is designed with this method it will crack in slabs; distances between the cracks are several meters, even tens of meters. This cracking is just tolerated or it is prevented with thick asphalt layers.

b) When the design is made on the basis of micro-cracking the tensile stresses are directed at asphalt layers, and the cement-treated layer functions on its compressive strength. This method is suitable for subbases.

c) In many countries, however, a third way of approach is chosen. It is supposed that although cement-treated materials do not have sufficient tensile strength and although they will crack also due to traffic loads, this cracking does not decrease the ability of a layer to bear and distribute these traffic loads and does not shorten its life period. This intermediate way of approach is considered possible owing to the good experiences received in different countries of the durability of cement-treated structures. With this method it is possible to exploit the superior compressive strength properties of cement-treated materials near the surface of the road. The cracking is tolerated and efforts are being made to control it by curing or by means of several layers of asphalt.

A 412 Design of cement-treated layers

In general, the total structure of a road must be so designed that it is sufficiently rigid to be able to resist the stresses caused by dynamic traffic loads and to offer a good service level of long duration. The task of structural courses is to distribute the stresses from the road surface to the subgrade so that the allowed stresses of the subgrade won't be exceeded and also to restrict frost heave and to even out differential frost heave. In the design the critical factors are the vertical compressive stress on the subgrade and the tensile stress on the lower surface of bound courses, (Figure A-3).

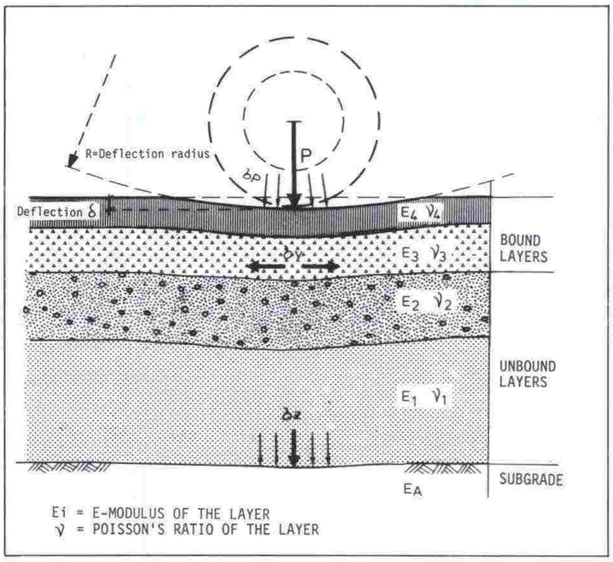


FIGURE A-3. Deflection and critical stresses of a road structure

Design of structural courses can be performed as a multilayer construction on the basis of the stress - strain dependence on different materials caused by dynamic traffic loads. It is necessary to presume that the materials are flexible, linearly and elastically behaving; also many other simplifying assumptions are made, /6,7/. In addition to the strength of the subgrade and to traffic loads, the internal modulus of elasticity and the so called Poisson-factor (Figure A-3) must be defined as starting values, after which the stress condition of the chosen structural alternatives can be checked with computer programs.

In Finland the structural design of a pavement construction is based on the enclosed Odemark design equation:

$$E_y = \frac{E_A}{1 - \frac{1}{\sqrt{1 + 0{,}81\left(\frac{h}{150}\right)^2}} \frac{E_A}{E} + \frac{1}{\sqrt{1 + 0{,}81\left(\frac{h}{150}\right)^2 \left(\frac{E}{E_A}\right)^{2/3}}}} \quad [MN/m^2]$$

E_y is the bearing capacity on top of the layer surface to be measured MN/m^2

E_A is the bearing capacity under the layer to be measured, MN/m^2

h is the thickness of the layer to be measured, mm

E is the E-modulus of the layer to be measured, MN/m^2

The design is performed step by step and course by course from the subgrade upwards and by choosing layer thicknesses and materials so that the target load-carrying capacity will be reached.

The corresponding elasticity assumptions can be made for unbound and cement-treated materials and in a design process any course can be replaced by a cement-treated layer.

In this way the good strength properties of a cement-treated layer are taken into account in the design of the total structure. It is only a question of what are the material qualities (E-modulus) of cement-treated pavements that should be taken into the counting process. It is true that the E-modulus of cement-treated materials is considered to vary as much as 1500 - 15000 MN/m^2 depending on materials, proportioning and the success of the work. In Finland the following modulus values are commonly used E = 2000 - 2500 MN/m^2, (Figure A-4). In 1973 on the testroad of Palojärvi-Olkkala an E-modulus of 4500 MN/m^2 was used for cement-treated materials in a subbase course (strength 2-3 MN/m^2) and an E-modulus of 7000 MN/m^2 for cement-treated materials in a base-course (strength 3-6 MN/m^2), /8, 9/. It would be

Material	E-modulus MN/m²
Concrete	30 000
Rcc (roller compacted concrete)	7 500 (preliminary)
Asphalt concrete	1 500...2 500
Oil gravel	350
Bitumen gravel	2 500
Grouted macadam	700
Non-cohesive soil reinforced with cement	2 000...2 500
Cohesive soil reinforced with lime (short-term)	(200...400)
Non-frost-susceptible crushed aggregate (the grading envelope of the subbase or base course)	200...350
Gravel and gravelly sand (the grading envelope of the subbase course)	150...280
Non-frost-susceptible sand (the grading envelope of the filter course)	30...100
Frost-susceptible crushed aggregate or gravelly till	
- in an embankment	150
- in a cut	100

FIGURE A-4. The E-modulus of the pavement structure materials in the Finnish design practice /40/

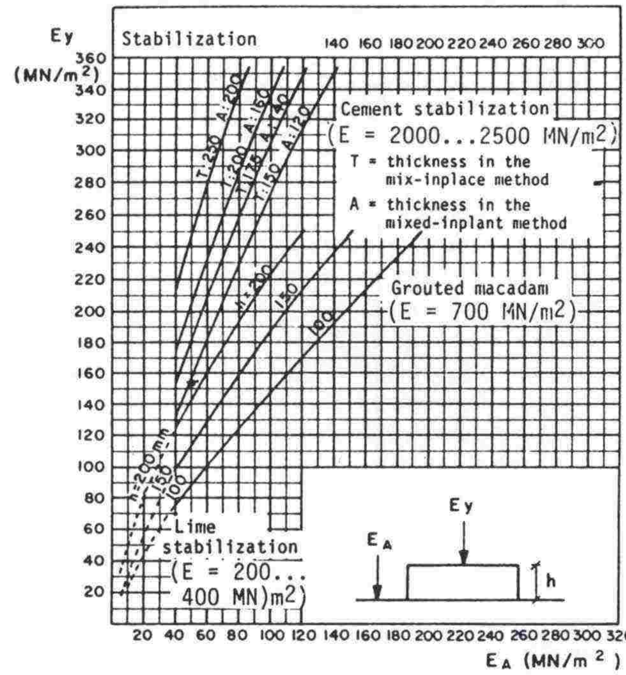


FIGURE A-5. Thickness design of cement-treated layers according to the instructions of the Road and Waterways Administration

appropriate to use higher strength and E-modulus values than those we are used to.

In practice the layer thicknesses vary between 12 - 25 cm. For example in the United States and in Finland a thickness of 12 cm is considered an absolute minimum. A thickness of 15 - 20 cm is most common. In Norway cement-treated layers of 10 cm on the base-course have been used, /10/.

Various countries have their own specifications with standard structures or with design tables, on the basis of which also the correct cement-treated alternative can be studied without the laborious counting process. This is the case also in Finland; the standard specifications by the Road and Waterways Administration from 1985 /11/ contain both a dimensioning diagramme, Figure A-5, and standard structures, an example of which is shown in Figure A-6. The equivalences in Figure A-7 show the effectiveness of cement-treated materials in material economy. Certain approaches to pavement design abroad are handled in chapter A 61. A thorough summary of the empirical and analytical approaches to pavement design can be read e.g. in the newly published book Cement-Treated Pavements by R.I.T Williams. /1/

A 413 Components and proportioning of cement-treated mix

Aggregates

Cement-treated materials were originally developed into materials which would make it possible to use marginal aggregates in pavements. In principle all non-humus soil types which are rougher than silt are proved suitable for binding with cement. Tolerance grading is comparatively wide, Figure A-8. It covers sand, gravel and moraine. The basic requirement is that the amount of particles smaller than 0,074 mm remains under

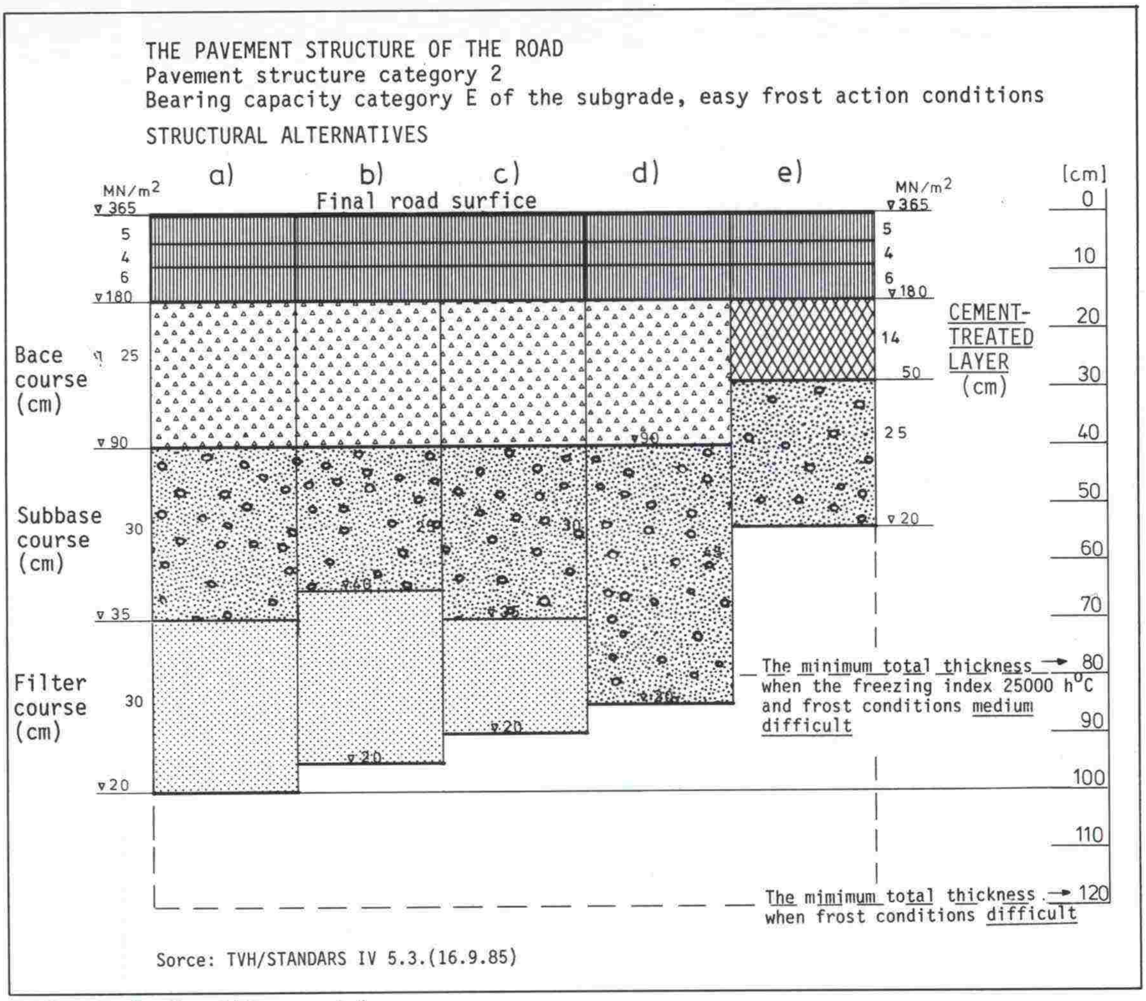


FIGURE A-6. Alternative pavement structures, the example from the instructions of the Road and Waterways Administration

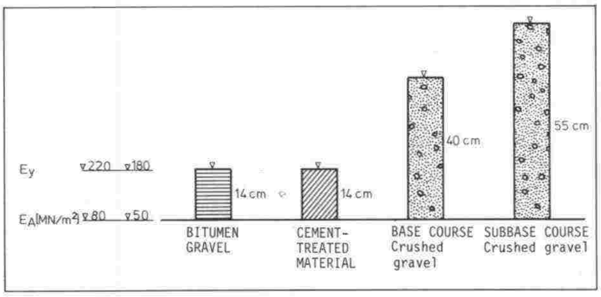


FIGURE A-7. Correspondence of different pavement structure materials according to the instructions of Road and Waterways Administration /40/

30...35 % and that there are no stones bigger than 60 mm in the material.

Aggregates can be frost-susceptible, sandy or opengraded in which case addition of cement compensates the impurity in aggregates. Thus it is possible in principle to obtain homogeneous structural strength with cement-treatment inspite of the original aggregates.

Different countries have different grading envelopes owing to the availability of aggregates and to the purpose of cement-treated materials, Figure A-9, but in general aggregates used in different countries are within the scope of the above definition.

In addition to natural aggregates it has become more and more general to use industrial wastes and by-products, such as different kinds of slags, as an aggregate for cement-treated materials. Also old pavement materials are used for recycling with cement.

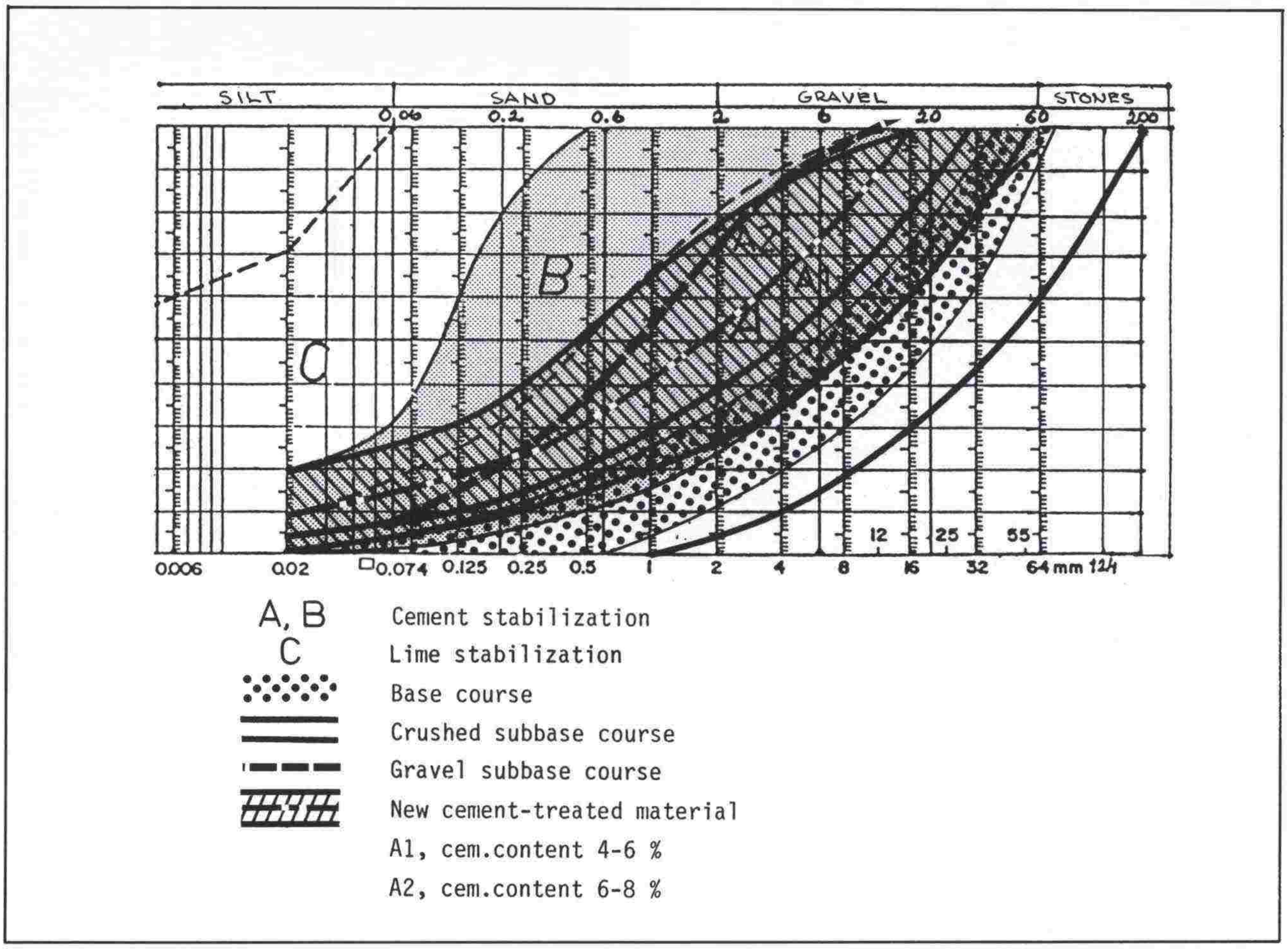


FIGURE A-8. Finnish grading envelopes for aggregates to be stabilized

In Finland several investigations and tests of the use of moraine as an aggregate for cement-treated materials have been made in the 1980's /12, 13, 14, 15, 61/.

When erosion, moisture and frost resistance are emphasized as structural properties of cement-treated materials, wide grading envelopes contribute to obtaining profitable structural courses to various parts of the pavement /16/. This concerns base and subbase courses on roads with less heavy traffic, but principally only subbase courses on roads with very dense heavy traffic.

When the compressive and flexural strength properties of cement-treated materials are emphasized - as must be done when dealing with roads with heavy traffic- stricter requirements have to be made on grading of aggregates /17, 18, 19/. Primarily on the basis of Central European experiences and investigations "a new cement-treated material" has been developed. The requirements for aggregates to be used in this are purity, strength and well-graded curvature.

In the norms of various countries the grading envelope usually corresponds to the grading envelopes of unbound base and subbase courses, however, the maximum grain size for cement-treated materials is 50 mm, as examples the grading requirements of the Finnish (Figure A-8) and German norms (Figure A-9).

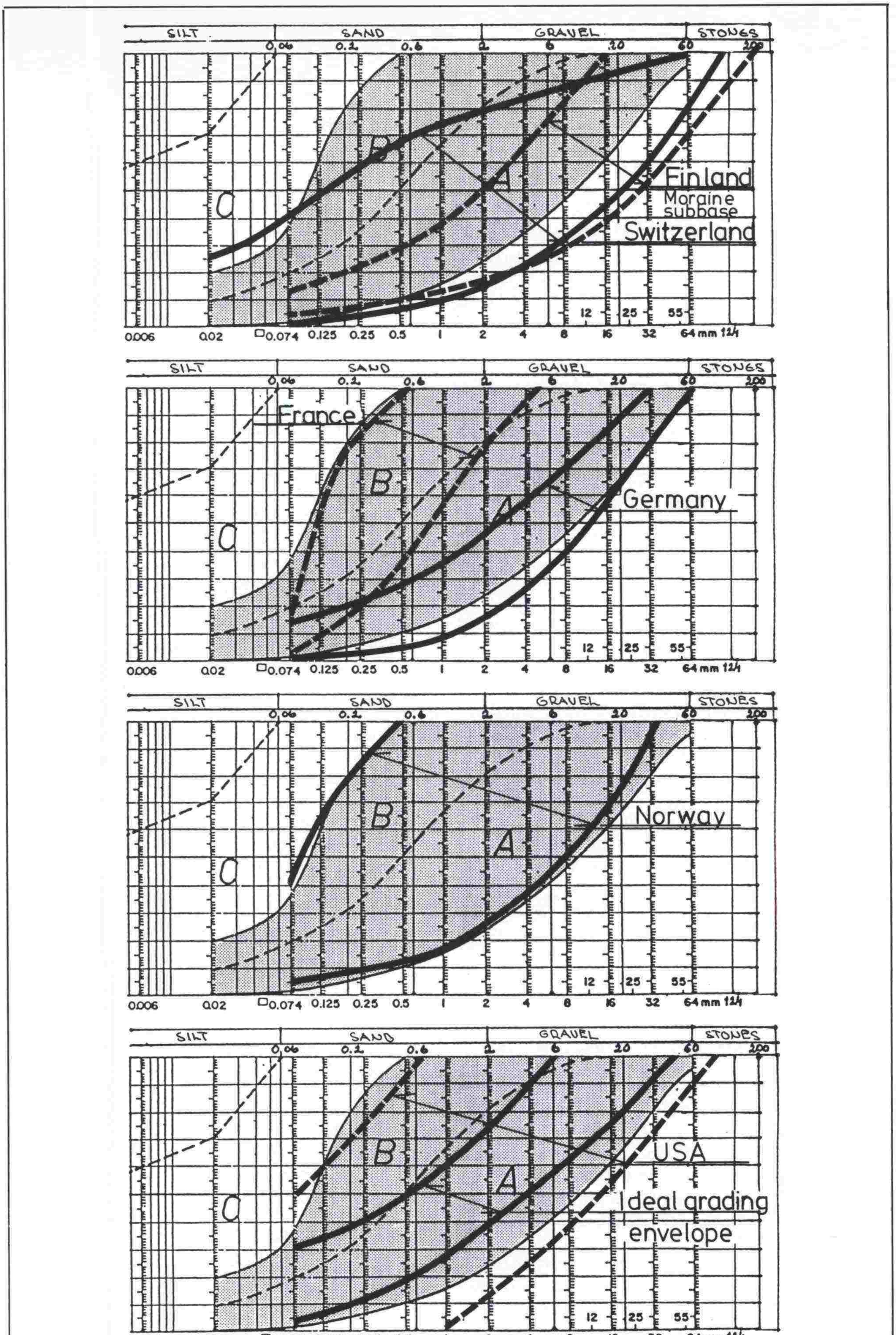


FIGURE A-9. Grading envelopes of cement-treated materials in different countries (A, B, C are the Finnish grading envelopes for lime and cement stabilization)

The greatest dry densities and the greatest strengths of cement-treated materials can be obtained with a grading curvature the middle part of which touches the lower boundary curve, but both ends approaches the upper boundary curve.

According to the Norwegian recommendation the total amount of fines (< 0,074 mm) and cement in cement-treated materials should be at least 10 %, /10/. On the other hand, if the amount of fines is more than 15 % cement-treated materials are difficult to handle. If aggregates are of crushed material- especially the amount which corresponds to the grading of sand - the best strengths are obtained.

Cement

Ordinary Portland cement is the binder most frequently used in cement-treated materials. In countries as Belgium, France and Spain where supply of hydraulic binders, blastfurnace slag and flyash, is great, cement-treated materials are made using these binders. Thus the amount of cement may be only 15...40 %, /20/. Also in Finland the use of blastfurnace slag is accepted but then the amount of cement must be at least 30 %, /21/.

In many countries blastfurnace slags are not favoured due to slow developing of strength. When 70 - 80 % of the 28-day compressive strength is obtained with Portland cement at the age of 7 days, the corresponding figures with slag mixes are 20 - 30 %. A good initial strength is considered important for cement-treated materials because cement-treated layers can only seldom be isolated from traffic so that a slow strength development can be taken into account. If this strength development is disturbed in its initial phase it may be fatal for the final strength.

On the other hand, use of slags allows a longer working time with fresh mixes, and the fully developed (90 days) final strengths are equivalent with Portland cement or even better.

Additives

Use of additives in cement-treated materials is not yet very common. The slowing effect of humus in the hardening process of cement can be neutralized with lime powder (1 - 2 % of the dry weight of aggregates). The same effect can be obtained more profitably by increasing the amount of cement with approximately 1 percentage unit. However, humus is an unpredictable factor and it is recommended to avoid aggregates which contain humus.

The setting reaction can be accelerated for example with calcium chloride (0,3 - 0,8 % of the dry weight of aggregates). In some cases using a retarder may be necessary to increase the processing time. The binding time is generally determined by the choice of a correct binder, not by the use of additives. More detailed investigations of additives have been made in France. They are also more frequently used there.

Mix design

Content of cement, water and aggregates in cement-treated mixes depends on the quality and grading of aggregates but also on the goals set for the material. Weather resistance is the most important factor in the United States and in Switzerland, and the proportioning is there defined with preliminary tests to correspond to this goal. A weather resistance goal can be - depending on the method- the maximum weight loss or the allowed maximum change in the

Unified Soil Classification*	Usual Range in cement requirement** percent by vol.	percent by wt.	Estimated cement content and that used in moisture-density test, percent by weight	Cement contents for wet-dry and freeze-thaw tests, percent by weight
GW, GP, GM, SW, SP, SM	5 - 7	3 - 5	5	3 - 5 - 7
GM, GP, SM, SP	7 - 9	5 - 8	6	4 - 6 - 8
GM, GC, SM, SC	7 -10	5 - 9	7	5 - 7 - 9
SP	8 - 12	7 - 11	9	7 - 9 -11
CL, ML	8 - 12	7 - 12	10	8 -10 -12
ML, MH, CH	8 - 12	8 - 13	10	8 -10 -12
CL, CH	10 - 14	9 - 15	12	10 -12 -14
OH, MH, CH	10 - 14	10 - 16	13	11 -13 -15

*based on correlation presented by Air Force

**for most A horizon soils the cement should be increased 4 percentage points, if the soil is dark grey to grey, and 6 percentage points if the soil is black

C clay S sand
M moraine O organic
G gravel

FIGURE A-10. Range of cement content in different aggregates according to an American instruction /2/

length of the sample after certain freezing/melting or drainage/saturation tests /2, 22/. Figure A-10 gives us an example of the American proportioning process.

In most European countries, however, cement-treated materials have the compressive strength as the top goal (Figure A-11) and the cement amount required is defined with preliminary tests using actual materials, /5/.

A preliminary test arrangement can be as shown in Figure A-12. With the improved Proctor test the optimum water content and the maximum dry density of a cement-aggregate mix will be defined with three different amounts of cement. Then the cylinders, which give the maximum dry densities, will be compressed at the age of 7 days and by the help of the compressive strength results estimates will be made with which amount of cement the target strength will be obtained, /23, 24/.

As cement and its transportation form approximately one half of the cement-treatment costs it is necessary both technically and economically to define beforehand the optimum content of cement. There are pre-calculated charts based on grading values available, but their use is not fully reliable. However, it is possible to present average values as in Figure A-10.

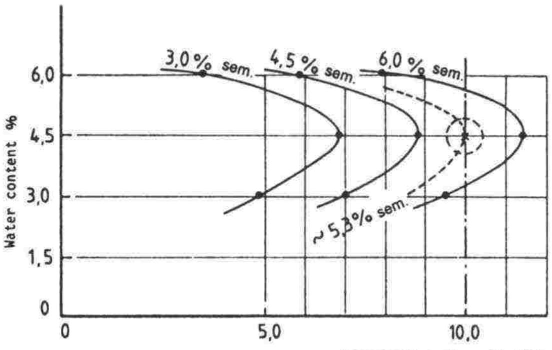
In Finland the strength target in base courses is 5 - 6 MN/m^2 at the age of 7 days and in subbase courses 3 - 5 MN/m^2. The strength target in base courses may be increased up till 10 MPa in the near future. The cement content varies between

Cement content (% on dry aggregate) and unconfined compressive strength (MPa)

Country	graded gravel/ coarse aggregate (I lk maabet.)		lean concrete (laihabet.)		sand-cement (II lk maabet.)		cement content	compressive strength	
	cement content % sem.pit	compr. strength MPa pur.luj	cement content % sem.pit	compr. strength MPa pur.luj	cement content % sem.pit	compr. strength MPa pur.luj.	remarks h uom.	after days vrk	remarks huom.
Australia	3-4	1-2	6	3	4-7	2-3		7	varies between states and aggregate used
Australia				7		7		28	
Austria	4½ˣ	3			5ˣ	0.7	local experience: min 5%	7	quality control
Austria		5				1		28	only for approval test
Belgium	2½-4		4-6	4				90	statistic minimum mean - 2 stand. deviation (cored, 0.15 m :)
Brazil			4-9						
Canada	7	3.5			5-7½	1.7-2.4	sand-silt: 10-11%.	7	usually 2-3 MPa:
Canada						3.4		28	variations per provinc
China	5-6	3-4						7	
Czechoslovakia	5-7	2.5-3.5	4½-6	7-12	6-8		silt: 8-9%	7	motorways
Czechoslovakia		1.8-2.5		6-11				7	otherways
Denmark	4ˣ	5	10ˣ	10	6½ˣ	3			statistic requirements
FDR Germany	4-6	7-12	6½-10ˣ			6	e.g. gravelly sand 0/32; min 3%; sand 0/2 mm: min 10%	28	0.15 m o, h = 0.125 m
France	3½		7½-8		5-7				
France	4.5			10.0				7	average of 10 } cubes
United Kingdom	2.5			6.5					individual } cubes
Italy	2½-3½	2.5-4.5					for new layers	7	motorways
Italy	3½-5	3.0-7.0					for recycling	7	other roads, airports
Japan	3-5	3			4-6	3			base } target value
Japan		1				1			subbase } target value
Netherlands					8	5	mean value, depends on sand	28	Proctor 0.10 m o Wopt-2 for detn. of cement content.
Netherlands						1.5		90	Statistic min. = mean - 1.3 stand. dev. for n=20 (cored, 0.10 m :)
Norway	5-8	3-4						7	Recommended: min. 5 MPa
Holland	2-4	1.6-2.2		3.5-5.5	8.10	1.0-1.6		7	
Holland		2.5-5.0		6.0-9.0		1.5-2.5		28	
Spain	3-4½	6			3-6	2.5		7	In practice 4.5-7 and 2.0-3.5 MPa
Spain		9						90	
Sweden	4-9	5.0			4-9		gravel or sand 6.3% mean value	7	In practice mean 6.6 (3.6-10.0) MPa
Switzerland	3-4½ˣ	2.5						7	
USSR						4-10ˣˣ			
Yugoslavia	3-4	2.0-5.5					examples; sometimes higher to satisfy frost criteria	7	0.15 m o, h = 0.15
Yugoslavia	3-4	3.0-6.5						28	5.0 as target value

ˣ Calculated from data in kg/m^3 with density 2.0 and 1.8 for sand-cement
ˣˣ Used for impregnation of crushed stone bases.

FIGURE A-11. Cement content and compressive strength of cement-treated materials in different countries /5/



- test cylinders are made by a Proctor mould with three different cement contents and with three water contents; compacted at the age of 7 days
- the cement amount needed for the required strength is interpolated

FIGURE A-12. An example to find out the needed water and cement amount /4,24/

4 and 8 % in the grading envelopes A1 and A2 and between 7 and 10 % in the grading envelope B calculated from the maximum dry density of aggregates, /4, 25/.

With a cement content of 3 - 5 % the required compressive strengths are reached when using high-quality aggregates, but for many reasons a cement content of 4 % is considered the minimum value in Finland and also in other countries. This minimum value secures the erosion and frost resistance and decreases risks caused by the working quality.

A 42 CONSTRUCTION

A 421 Major demands for the work

It is an advantage of cement-treated materials that they can be processed fresh and formed on the road in a way of loose materials with conventional equipment and also that the strength development takes place only afterwards. The addition of cement as such is not sufficient to improve load-carrying properties of an aggregate. It is essential that the work will be succesfully performed.

Because the amount of cement in cement-treated materials is so small the mixing and homogeneity of the material are of vital importance. If cement-treated materials remain loose they lack strength. That is why it is important that the compaction has been made efficiently. Insufficient curing, again, spoils the strength development and increases cracking.

In order to achieve the planned targets it is necessary that not only the work control but the working process as a whole will be accomplished with such equipment and methods that a homogeneous cement-treated layer will be obtained. The objects are:

- a correct water content (near the optimum water content)
- a correct cement content (evenly in the material)
- a sufficient degree of compaction (generally at least 97 % with the improved Proctor test of the maximum dry density)
- favourable temperature and humidity conditions for the strength development in the curing time (working temperature more than +5 C, a good curing material)
- the pavement must be protected against traffic for long enough a time

A 422 Working methods

The well-known working methods of cement-treated layers are the mix-in-place and mixed-in-plant methods. The mix-in-place method is older and special equipment for the different working phases has been developed especially in the United States, /2/.

The mix-in-place method is most profitable when aggregate materials available on a road can be used. Although the quality deviation of the material will be greater with this method, this is regarded as a fully parallel working method to the mixed-in-plant method in many countries, especially in the United States.

The mixed-in-plant method is considered profitable when high-quality aggregates transported from elsewhere are used. According to the norms in many countries, e.g. Norway, Denmark, Germany and Spain, the mixed-in-plant method is, because of the evener quality, the only acceptable method for cement-treated base courses /5, 26, 27/.

TABLE A-1. Comparision of different working methods of cement-treated layers

ISSUE	MIX-IN-PLACE METHOD	MIXED-IN-PLANT METHOD
Quality	Quality level more unhomogeneous Greater deviation	A good quality level Small deviation
Price	Often less expensive	Competitive, when the hauling distance of the mix is short
Size of the project	Suitable for small projects	Often requires a relatively great project
Performance of the work	Many working phases in the field	Less working phases
Special equipment	Mixer, cement spreader	Concrete plant, paver
Base	Aggregates of the layer function as a base	Calls for an even, bearing base for the paver
Use of material	Can utilize existing aggregates	Aggregates transported from elsewhere
Handling of the mix	Sprinkling with water and formation with a grader possible in the finishing phase	The spread layer is not usually formed any more, requires an even base
Effective working hours (not more than 2 h)	More time for compaction before the setting be- gins	Shorter working hours due to the hauling of the mix
Applicability	Always when using aggre- gates of old roads	On base courses of heavily trafficked asphalt-paved roads
	In other cases the working method is to be chosen according to local conditions, available equipment or on the basis of price comparisions.	

In Finland so far 2/3 of the cement stabilization work has been made using the mix-in-place method and 1/3 with the mixed-in-plant-method.

Different working methods and their suitability with advantages and disadvantages are dealt with more thoroughly in the inclosed table (Table A-1).

A 423 Working phases and equipment

The essential working phases in cement treatment are /28/:

<u>Mix-in-place</u>

- preliminary work: transport of additional material, removal of stones, loosening of soil, sprinkling with water etc.
- spreading cement
- mixing
- finishing
- compacting
- curing

<u>Mixed-in-plant</u>

- preliminary work: supply of
- aggregates, erection of a concrete plant, compaction and forming of the base etc.
- mixing and transport
- spreading
- compacting
- curing

In the mix-in-place method removal of stones and loosening of soil are demanding working phases and the homogeneity of the final result depends greatly on them. Spreading of cement can be made with bags if it is a question of a smaller work. When dealing with greater areas the spreading work can be made either with lime spreaders used in farm works or with special equipment (Figure A-13). The spreading precision should be good and the needed amount of cement should be spread at the same time. In these respects every equipment used nowadays is insecure.

The mixing shall be made using special mixers. In Finland light mixers actually used in agricultural works have been common. Their mixing capacity has been adequate only for thin soil-cement layers. Also new types of mixing equipment have been developed in Finland, an example of them a plough-like mixer.

The heavier equipment lately introduced in Finland has given us good experiences, Figure A-14. The mixing results have been good already after the first drive when the layer thickness has been 15 - 17 cm. There is also still heavier equipment which has been used abroad. However, their economical use provides big construction sites and greater pavement thicknesses.

The WINDROW-method, /2/ which is well-known in the United States, is a special method where the aggregates are collected or laid as a longitudinal windrow. Cement is spread on the flat top of the windrow and the material is mixed and spread with a special mixer, which picks up the material, mixes it and spreads again on the road.

In the mixed-in-plant method the compaction and forming of the subbase is an important preliminary working phase,

FIGURE A-13. Laying cement with a Panien spreader (bought to Finland in 1988)

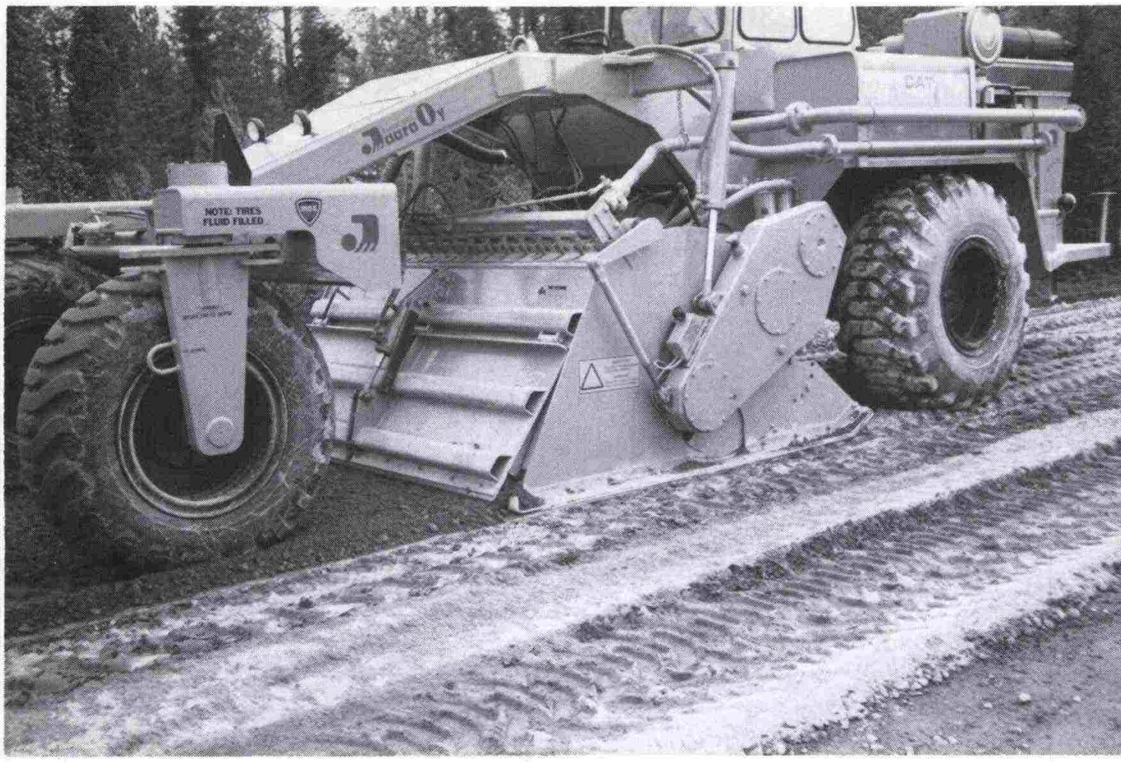


FIGURE A-14. A Cat RR 250 soil stabilizer in operation

FIGURE A-15. A site mixing plant used for producing cement-treated aggregate because the material is laid as a layer and the evenness of the surface cannot be finished after the laying process. The material is made and transported as in usual concrete production, the laying is made either with a drag which is fastened to the truck or with an asphalt paver. When cement-treated materials are used in base courses and the demand for evenness is great, it is necessary to use steering wire and automatic levelling in a paver. The manufacturing capacity of the material should be so great that the laying operation can proceed in a continuous manner. At the end of each day's work and during pauses longer than two hours a vertical construction joint as a full-depth transverse joint is made. The adjoining lanes shall be laid within two hours and so that the longitudial joint can be compacted before the setting of the soil-cement begins.

In both methods 97 % of the improved Proctor compaction is regarded as a proper compaction requirement in the whole cement-treated layer. This is possible when compaction is taking place in the optimum moisture conditions and by using heavy compaction equipment. The heavy vibratory rollers of 8 - 10 tons are the most reliable soil-cement rollers. To obtain the best possible result the suitability of the equipment should be tested with a test rolling. In a loose material compaction is started without vibration and it is completed with 4 - 6 rollings every time increasing the vibrating power. The use of heavy combination rollers (vibratory roller-rubber tyred roller) has become more and more popular in cement-treatment; a rubber tyred roller secures the compaction of the surface and gives a good finish. The compaction must be completed before the setting starts, within two hours after mixing the cement at the latest. An efficient compaction must not lead to short thicknesses; a sufficient compaction margin must be taken into account in the loose thickness of the layer. On the soil-cement surface there must not be disintegrated areas; they must be mixed and compacted again.

Except compaction also curing has an important roll in the strength development and crack forming of cement-treated pavements. The layer must be kept moist and it must be protected from traffic. The surface can be protected by sprinkling it with water or with special curing materials. The most reliable method, however, is protection with bitumen emulsion (0,8 - 1,0 kg/m^2) which is done immediately after the compaction on a fresh soil-cement surface; if the surface is dry it can be moistened before spreading the emulsion.

Alternatively, the first asphalt layer can be spread on a fresh soil-cement surface; according to a Dutch experience this method secures best that cracking is only harmless hair cracking.

Protection from traffic is all the more important the greater flexural strength the layer is expected to have, in other words the higher in the structure the layer is situated. It is generally regarded that when using normal cement a cement-treated pavement can be opened to a lighter traffic after 3 days and to a heavy traffic after 7 days. If the road must be opened to traffic earlier, the cement-treated pavement should be protected with asphalt or with a layer of crushed stones (> 10 cm). The risks are estimated case by case for excample using the Swedish approaching method as stated in Figure A-16. In the specifications by the Road and Waterways Administration light vehicle traffic can be allowed after 1 day and heavy traffic after three days on a base course. According to the specs the strength should be > 2 MN/m^2 before opening the road to traffic.

If the asphalt is laid directly on a cement-treated material, an emulsion treatment with chippings is often necessary to secure the tack coat of the asphalt and cement-treated pavement. Surface treatment decreases also reflection of cracks into the asphalt surface.

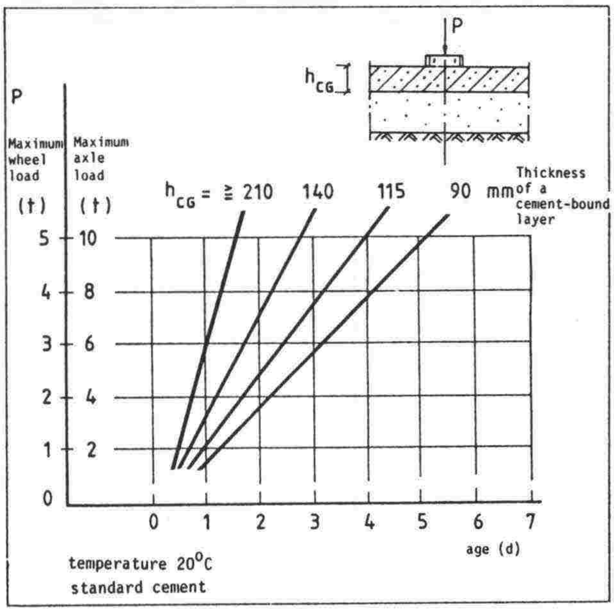


FIGURE A-16. Axle-load limits for fresh cement-treated layers according to Swedish instructions /23, 24/

A 424 Quality control

Quality control, which is performed during the cement-treatment, surveys the grading, water and cement content and compaction degree of aggregates. The final result is judged on the basis of the compressive strength. It is possible to test also frost resistance (frost susceptibility, water resistance) and flexural strength afterwards. Laboratory instructions and specifications contain requirements for test methods and sampling equipment and they do not differ much in different countries. Only in attitudes to compressive strength and frost resistance there may be differences.

Different countries set different targets to compressive strength and it is tested with different methods, in Table A-2 testing arrangements and strength targets in Western Germany, Sweden and Finland are presented. These differences should be recognized when comparing different compressive strength values.

TABLE A-2. Definition of the compressive strength of cement-treated materials in Germany, Sweden and Finland

	Diam. of the sample	Height of the sample	Procedure	Smoothening sample ends	Loading age (days)	Loading speed	Compress. str. MPa	Note
Germany	150	125	vibrating table + bulk weight < 12 kg on the sample	gypsum or quick cement mortar 2 days before the test	28	0,1 MPa/s	7 - 12	Also a cylinder of 150x300 mm available; then the strength values are ab. 30 % smaller
Sweden	152	130 ± 3	vibrating with a Kango hammer	cement mortar or rubber sheet	7	incr. in the depression 0,03 mm/s	5 - 10	When levelling with a rubber sheet the strength values are about 50 % smaller
Finland	152 (102)	114 (117)	tampering with a Proctor hammer	cement mortar or sulphur	7	0,14 MPa/s	4 - 6	

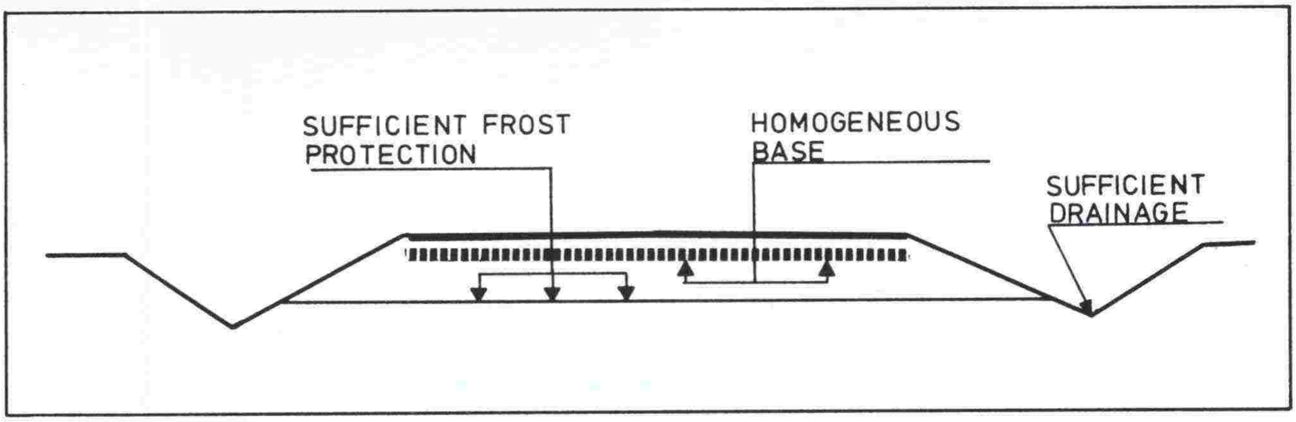


FIGURE A-17. External qualifications for the durability of cement-treated layers

In Germany both compressive strength and frost resistance are part of the quality control performed during the work, /26/; in many countries (USA, Holland etc.) compressive strength is tested only with preliminary tests; frost resistance and the weathering effect caused by moisture variations are followed in the quality control, /29/.

No measurable frost resistance requirement has been set for cement-treated pavements in Scandinavia. It is thought that compressive strength is related to frost resistance. It is provided that cement-treated pavements are absolutely non-frost susceptible. In Germany a change of < 1 o/oo in height in a frozen Proctor sample after 12 freezing cycles is required, /27/.

Flexural strength is generally defined as a splitting strength of the cylinder samples; also specially made beams are used.

As a rule, it is quite difficult to make samples for testing, when it is a question of comparatively dry materials. The present methods are not well adapted to the field control of cement-treated materials. New testing methods are to be developed in different parts of the world, especially concerning rcc (roller compacted concrete). Taking 7- or 28-day core samples directly from a road and testing them may be enough to follow up the final result of cement-treatment. This would also be a an easy method for thickness control. This is the practice for example in Canada but there it concerns lean-concrete.

A 43 QUALIFICATIONS FOR THE DURABILITY OF THE CEMENT-TREATED LAYER

A cement-treated layer is expected to have a good lasting durability. Correct design, good materials or a professional construction are not enough. Such "external" factors as load bearing capacity of subbase, drainage or frost protection of the construction are essential as to durability, (Figure A-17).

A 431 Load-bearing capacity of the subbase

A sufficient load-bearing capacity of subbase on which the cement-treated layer is laid is important for two reasons:

- On a weak subbase a cement-treated pavement cannot efficiently be compacted and the strength remains low.
- On a weak subbase flexural stresses caused by traffic exceed the allowed values, the cement-treated pavement cracks and looses its rigidity.

The higher in the pavement structure the cement-treated layer will be situated the more important it is that the subbase has a sufficient load-bearing capacity all the year round. The weaker the load-bearing capacity the higher must the safety factor be when designing the thickness of a layer.

This minimum level of the spring load-bearing capacity of subbase, which cement-treated materials can be designed on, varies between 20...40 MPa (5...10 CBR) in different countries /2, 5/. According to the specifications by the Road and Waterways Administration the load-bearing requirements under cement-treated layers are at least 40...50 MPa, /21, 25/. When designing cement-treated layers for base courses on main roads the minimum requirements vary between 80...120 MPa in different countries, /5, 30/. This kind of a minimum value has not been defined in Finland.

A 432 Drainage of the structure

A road construction should be such that the cement-treated layer and the unbound layer immediately under it can be drained freely to side ditches or to subsurface drains on both sides of the road. This is important because

- a water saturated subbase loses some of its load-bearing capacity in the melting phase and then it does not support a cement-treated pavement sufficiently
- the frost resistance of a cement-treated pavement would suffer too much if the pavement "swims" in water during the freeze and thaw phase; the pavement cracks and loses its rigidity.

A 433 Frost protection

A cement-treated structure must be properly protected against frost; differential frost heave and the loss of load-bearing capacity during the spring must be levelled with sufficiently thick non-frost susceptible layers and with transition wedges. If this is not the case a cement-treated pavement made as a subbase or base course will be quickly distroyed and steep differences in frost heaves weaken the driving comfort. We have many bitter experiences of this in Scandinavia of the roads repaired with cement treatment in the 1960's and 1970's. In Finland cement-treated structures have been avoided if a total frost heave exceeds 100 mm or an uneven frost heave exceeds 50 mm, /25/.

When it is a question of an old road the frost damage and the need of transition wedges must be studied already in the design phase and the necessary change of materials should be made beforehand. As far as new roads are concerned the total thickness of a pavement can be defined on the basis of a proper frost heave design. Thin cement-treated layers which are laid either on a base or subbase course do not affect the total thickness of a pavement structure, /30/.

An increase in the structural rigidity can be considered a decreasing factor of thickness only if cement-treated layers are thick (> 25 cm) or if a cement-treated layer is placed as the lowest layer of a pavement structure (a sandwich structure), /22/. Even then the differential frost heaves must be levelled separately. Frost protection of a rigid pavement will be handled more closely in Chapter B 4.

A 5
USE OF CEMENT-TREATED MATERIALS IN DIFFERENT PARTS OF THE ROAD STRUCTURE

It has already become clear that cement-treated materials can be used in a road structure in different ways. Originally a cement-treated layer was used as a base for a concrete pavement or as an intermediate layer improving subgrade. Later cement-treated materials have been used in different layers of a mixed structure or they have been used to improve the load-bearing capacity of old roads. Different ways of use involve own targets and special features, which are presented here.

A 51 CEMENT-TREATED MATERIALS IN THE BASE COURSE (Figure A-18)

When cement-treated materials are used in a base course of an asphalt paved road the objects may be

- an increase in the load-bearing capacity compared with an unbound layer
- cost savings compared with a bitumen bound layer
- less deformation in the structure
- a longer lifetime of the structure

A combination of cement-treated materials and asphalt gives the upper part of a pavement structure a better rigidity than using only asphalt. This also contributes to a longer life period. With fluctuating price trends also the direct competitiveness of cement-treated pavements ranges compared with asphalt pavements. But because cement is well available and also made in Finland cement-treated materials are more and more often chosen for a base course. This is the case also in countries where thick asphalt layers

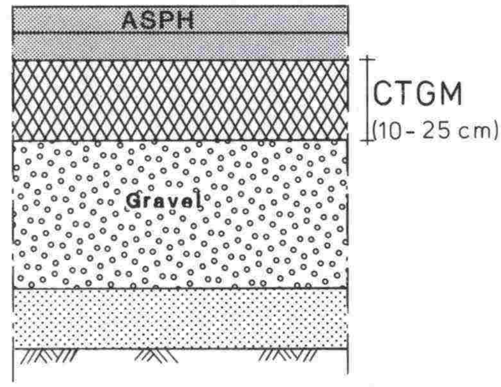


FIGURE A-18. Cement-treated materials in a base course

have been used more than in Finland. It is the base courses made of cement-treated materials that are of interest all over the world at the moment, /5/.

In a base course cement-treated materials are at their best; their strength qualifications can best be utilized there. But it is the base course that sets the greatest requirements for aggregates, proportioning, frost resistance, working methods and quality control. There is also a great risk for crack reflection into the surface of a pavement. However, many countries have had experiences that hair cracks on thin asphalts do not weaken the durability of a structure and do not shorten its life period; the damage is cosmetic.

In any case, due to a hair cracking risk thin asphalt layers (5 - 10 cm) on top of cement-treated layers are allowed only for inferior roads in most countries, /5, 30, 20/. It is general that minimum asphalt thicknesses of 10 - 20 cm are required on main roads. These requirements have weakened the actual competiveness of cement-treated pavements and accelerated investigations to avoid crack damage and to escape thick asphalt layers. In addition to the control of the cracking phenomenon itself (Chapter A 3) there are also promising methods to prevent reflection cracking, such as the use of rubberized bitumen in asphalt and different kinds of surface treatments with bitumen between cement-treated and asphalt layers.

When using thin asphalt pavements the adhesion of a cement-treated pavement and asphalt has to be secured with emulsion and surface treatment with chippings. The asphalt must also be as stabile as possible in order not to increase the deformation risk of asphalt because of cement-treated materials underneath.

A cement-bound base course can also be made of rcc, lean-concrete or of actual concrete. The strenghts are then greater than in cement-treated materials and cracking is controlled by sawing and crack sealing. Reflection of such joints up to asphalt surface is also considered difficult. However, good experiences have been received of jointing and sealing the asphalt surface and concrete base at the same places, (Figure A-19).

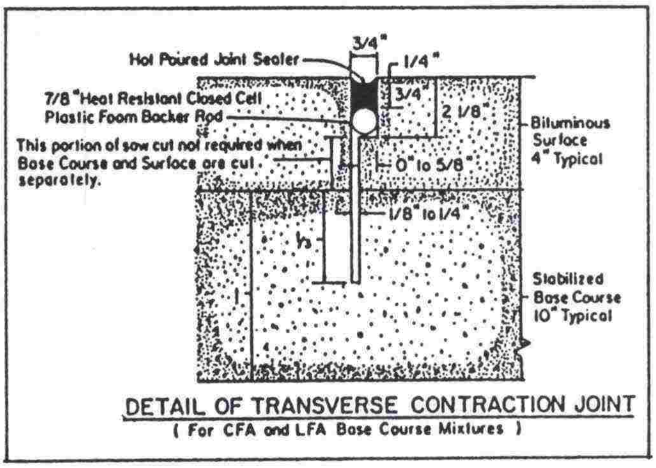


FIGURE A-19. Transverse contraction joints of mixed structures in an American design specification /57/

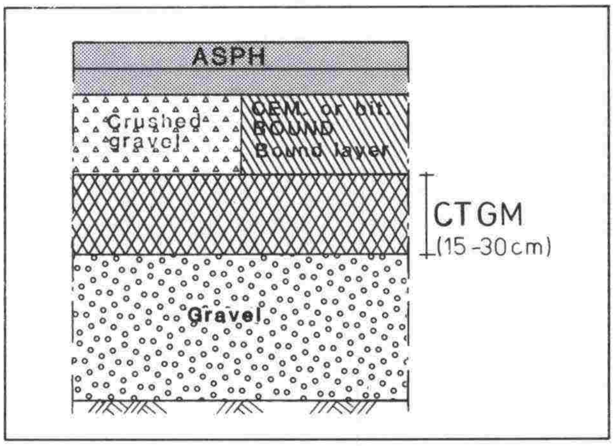


FIGURE A-20. Cement-treated materials in a subbase course

Base courses which are stronger than cement-treated layers- lean concrete, rcc - are mentioned in the norms of many countries, e.g. Spain, England, Ontario and many other states of USA, /5, 31/. They are of course used only on motorways with the heaviest traffic.

A 52 CEMENT-TREATED MATERIALS IN THE SUBBASE COURSE, (Figure A-20)

When designing cement-treated pavements for an upper part of a subbase the objects may be:

- thinner structural thicknesses and thus savings in materials and costs
- utilization of sandy or otherwise unfit aggregates
- increase of the total structural rigidity and thus longer life period.

In a subbase course cement-treated materials function in an "optimum situation"; effect on the total structural rigidity is efficient, requirements for materials and working precision are moderate and there is no risk for cracking of pavements. Strength requirements are also smaller than in a base course.

A proper design would provide that on a cement-treated subbase course a bound base course should be designed, in other words another cement-treated or asphalt layer. This is regularily the procedure on roads with heavy traffic, /20, 5/. In different countries it is a routine that there is a cement-treated subbase course on main roads and then a base course and pavements of bitumen; it is rare to make both subbase and base course with cement, /20/.

The use of an unbound base course (10...20 cm) on a cement-treated subbase is of interest for different reasons:

- Cracking risk of a pavement can be avoided.
- In uncertain frost conditions an intermediate layer evens out differential frost heave.
- Unbound material protects the fresh cement-treated layer from drying and traffic damage.
- It is easy to form the unbound material as a base for a pavement surface; the levelling requirements for cement-treated pavements become easier. Inspite of crushed gravel and crushed rock also opengraded macadam can be used.

From the point of view of the design this kind of a structure is "heretic". As to load-bearing qualifications a clearly weaker material is laid between cement-treated and asphalt layers. Also drainage of the intermediate unbound layer has aroused doubts. However, experiences in practice have been good in Finland and also in other countries, /5/. This "compromise" is considered good when traffic is not very heavy. France, Switzerland and Romania are testing this structure also on roads with heavy traffic.

A 53 CEMENT-TREATED MATERIALS IN LOWER PARTS OF THE SUBBASE (Figure A-21)

By building a cement-treated layer as an intermediate layer between a subgrade and the actual pavement layers many advantages can be reached:

- Compaction of the layers laid on top of a cement-treated layer will become easier and thus compaction requirements can be higher.
- Differential forst heave will be evened out; a cement-treated layer acts as a transition wedge, the total thickness of a pavement can be thinner. On a weakly load-bearing or compressible subgrade a cement-treated layer forms something like a matress and levels plastic deformations and settlement differences. On a very weakly bearing or watery subgrade compaction of a cement-bound layer won't succeed. In this case a lime stabilization of the subgrade can be made first.

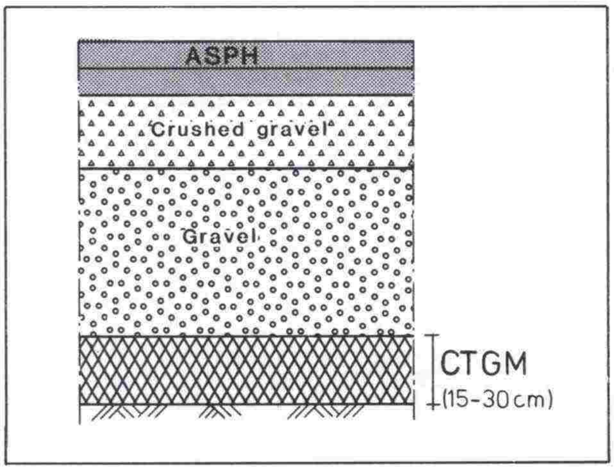


FIGURE A-21. Cement-treated materials on the surface of the road base

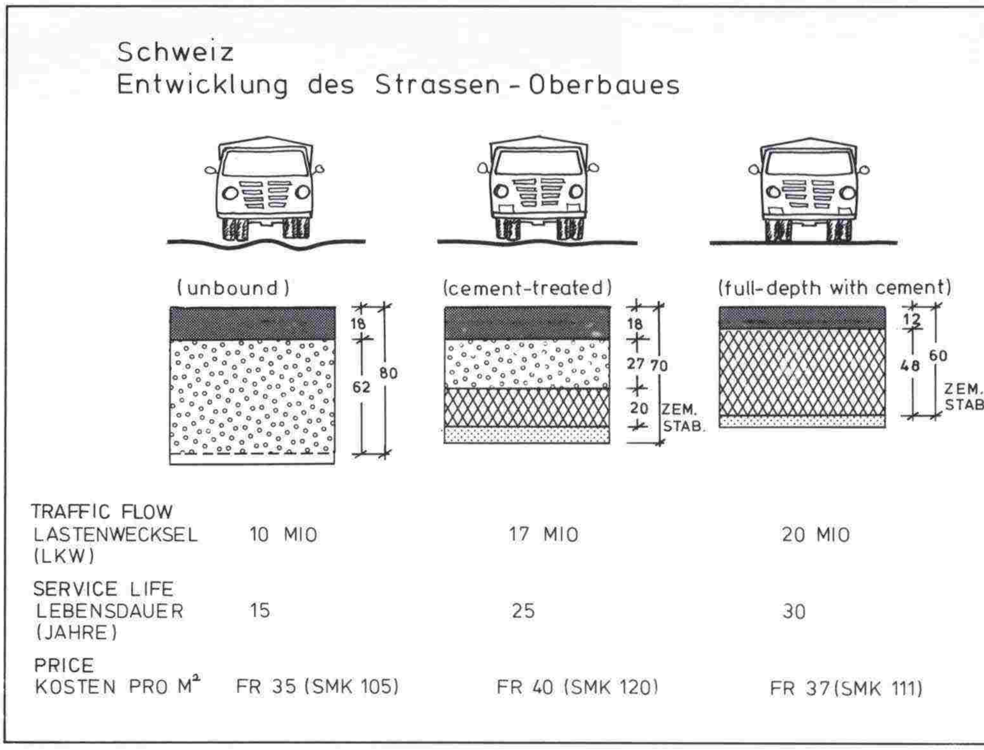


FIGURE A-22. Significance of cement-treated materials in the pavement structure according to a Swiss comparision /70/

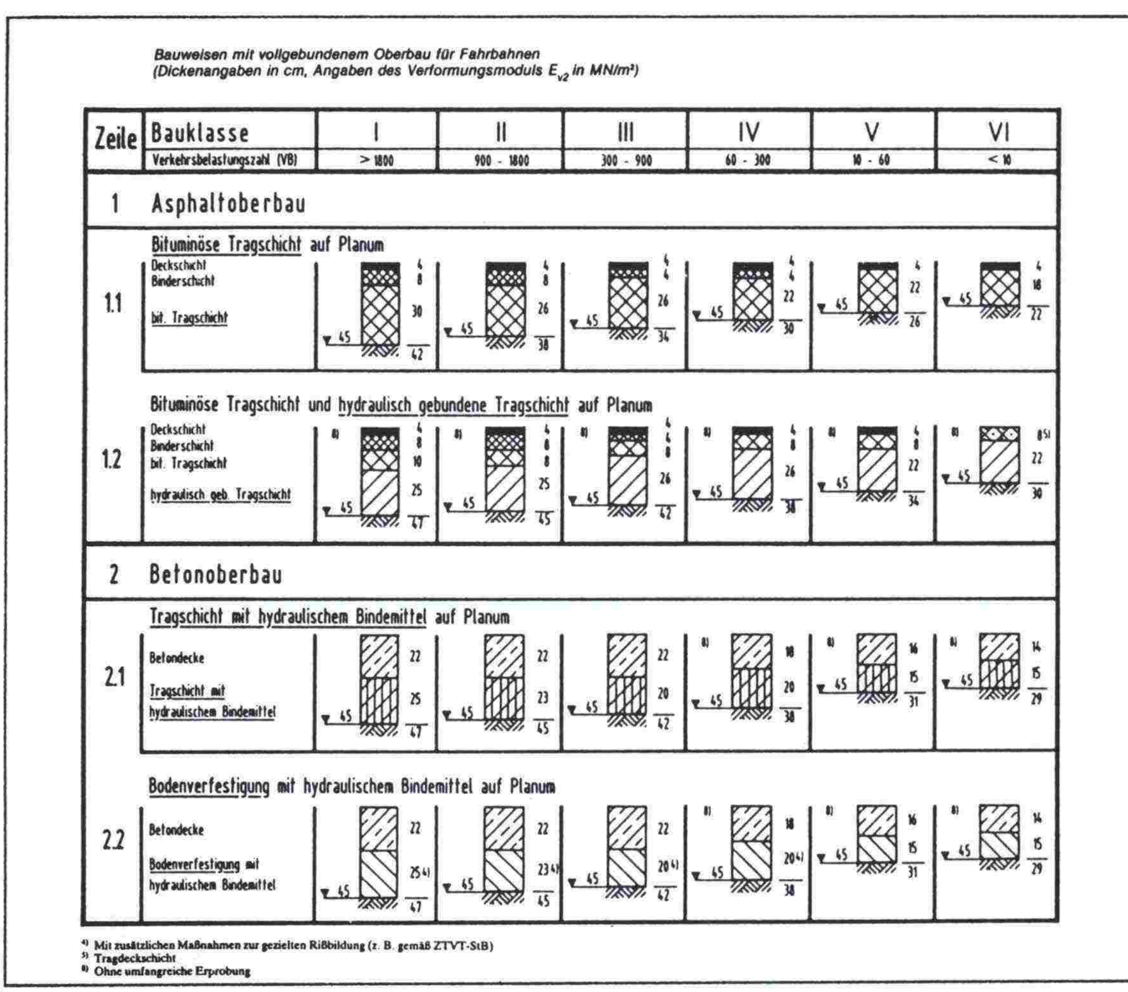


FIGURE A-23. Cement- and bitumen-bound full-depth structures in the German standards /30/

Of this 'sandwich' structure the Swiss have had good and long-term experiences, /32/, and now the method is known all over the world. The sandwich structure can be understood as a part of a pavement structure, but it can also be used to improve a subgrade as to the load-bearing capacity or settlements. The strength requirements of a cement-treated layer like this cannot be very high, because the layer is compacted on a weak base, strengths of 2 - 3 MPa are sufficient.

A 54 THICK CEMENT-TREATED LAYERS

When we combine the different cement-treated solutions presented in the previous chapters we more or less think of full-depth structures. Especially in Switzerland the design of cement-treated pavements is based on thickness rather than on strength. This full-depth alternative is included in the norms and in Switzerland this alternative is regarded as competitive both technically and economically. Figure A-22, /22/. Also in Germany a full-depth cement-treated structure has been accepted in the new norms, Figure A-23, /30/. Frost protection has been abandoned in these solutions and a suggestion has been made that a thick cement-treated layer stabilizes both frost heave and load-bearing differences. Transition wedges should be made when needed. This kind of a frost protection, however, looks suspicious to many researchers and so far the experiences are not convincing. In Finland where the frost lies deep, frost protection must still be made with sand, but it is possible even here to effectively design the remaining structure with cement-treated materials.

A 55 CEMENT-TREATED LAYER AS A BASE OF THE CONCRETE PAVEMENT (Figure A-24)

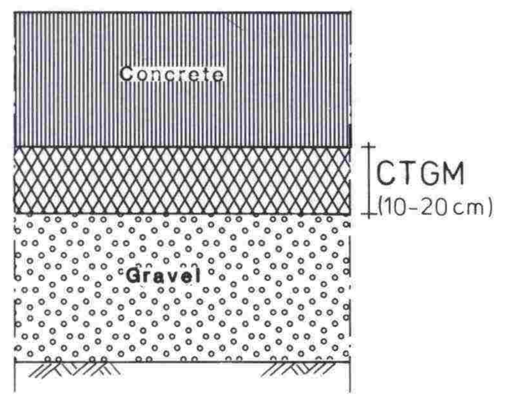


FIGURE A-24. Cement-treated layer as a base of a concrete pavement

A concrete pavement compensates also the base course so that a cement-treated layer which is built as a base of a concrete pavement is actually laid as a sub-base course. The views of a subbase course presented in chapter A 52 concern also the base of a concrete pavement. A cement-treated pavement is one alternative to fill the requirements (Chapter B 143) which a concrete pavement sets to its base.

As a base of the concrete pavement the main requirement of a cement-treated layer is not strength as such, but erosion and frost resistance and evenness, /22, 33/. In such a layer varying materials as to grading can be used, also different industrial wastes or crushed old pavements.

The maximum grain size is usually restricted to 20 mm. By using different cement contents and by performing the preliminary tests bases meeting the requirements can be built also of varying materials. The minimum thickness of a layer is generally 10 - 15 cm.

As to the quality of the work a cement-treated layer as a base of the concrete pavement must meet high requirements. The surface must be homogeneous, even and compact, especially if the concrete pavement is made with a slip-form paver. Unevenness and a varying absorbing capacity of moisture of the base may weaken the quality and evenness of the pavement. That is why it is often required that a cement-treated base for concrete pavements should be laid with heavy machinery with automatic levelling and possibilities to homogenizing of material.

If the strength of a cement-treated layer is too high compared with that of the concrete pavement, the hair cracks of a freely cracking cement-treated layer may reflect also into a concrete pavement as "wild" cracks. According to Canadian experiences this relation should be at least 3:1 in favour of a concrete pavement. In Finland we do not usually have this risk, because we use concrete with strength up to K50-K70. To avoid a cracking risk the binding between a soil-cement and a pavement is eliminated by handling the cement-treated surface with bitumen emulsion or with other isolating materials.

A cement-treated layer can also be designed as a part of a rigid pavement structure when both the pavement and the base act together, thus a complete binding must be provided between a cement-treated base and a concrete slab. This kind of design sets other requirements for the composition of a cement-treated material as stated before. These designing methods are exceptional - only test roads have been built - but especially in connection with rcc, utilization of the co-operation between a cement-treated base and a concrete pavement is of great interest.

A 56 CEMENT-TREATED PAVEMENTS IN THE IMPROVEMENT OF THE LOAD-BEARING CAPACITY OF OLD ROADS

When old road structures are improved the aim is usually to use the old structure as part of the new structure. The object is to restore the condition of the road and increase the load-bearing capacity with as little additional material and costs as possible. On certain conditions, the use of cement-treated pavements is a very suitable and profitable method when improving a structure /34/. In Figure A-25 there are some in principle different alternatives of the use of cement-treatment in rehabilitation of old road structures.

In order to succeed the quality of a cement-treated layer must meet the load-bearing capacity requirements, also the external factors mentioned in chapter A 43, a sufficient bearing capacity of a base, good drainage and a proper frost protection, must be in order. Lack of these conditions has often been the reason why a newly repaired road with cement-treated base or subbase has soon damaged again.

These kinds of bad experiences have been reported from all over Scandinavia, especially from Sweden, when cement-treated pavements and cement stabilization were used to improve unpaved road structures without taking sufficient care of the preserving conditions, /35/. On the basis of these experiences we know now that cement-treated pavements are not suitable for improvement of the load-bearing capacity of stony and unevenly frost-susceptible roads. Instead, if an old road has proper structure its damage can be repaired and the load-bearing capacity can be improved profitably with cement-treated layers and cement stabilization.

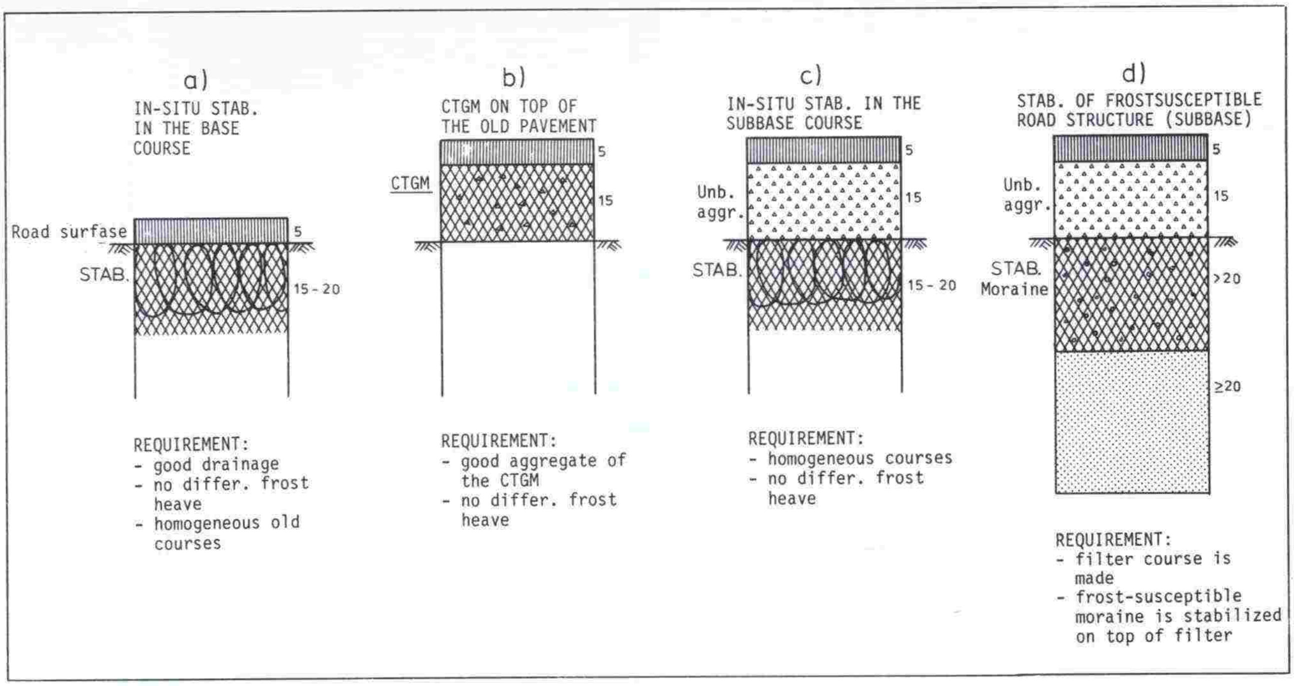


FIGURE A-25. Improvement methods of old roads when using cement-treated materials

In many countries it is a common practice to start the improvement of a road structure by milling the old damaged road surface. A homogenized layer can be left unbound or it can be stabilized with bitumen or cement. This homogenized layer forms a base for cement-treated, concrete or asphalt layers. Actually, the increase of the load-bearing capacity is undertaken by these bound layers.

A 6
EXPERIENCES OF CEMENT-TREATED PAVEMENTS

A 61 EXPERIENCES FROM ABROAD

Cement-treated pavements have been used long and all over the world. Each country has its own practice and tradition in using and designing cement-treated pavements. Many countries have also made thorough investigations on the durability of their cement-treated pavements. On the basis of own experiences specifications and norms have been drawn up for design and construction. Differing experiences and conditions have led to a diversified specs collections, Figure A-11, /5, 1/. Planning practice in different countries has been dealt with earlier in the report.

As an example of standard structures extracts from German, French and Swiss standards are presented, Figures A-26, A-27 and A-28. Not only the contents of the norms but also the actual adaption of cement-treated pavements differ in many countries. For example only some states in USA use cement-treated layers as a base of a concrete pavement, Figure A-29. Of the European countries only Denmark, England and German Democratic Republic join regularly a cement-treated base to a concrete pavement, in other countries also unbound and asphalt bases are in common use. In France a semi-rigid cement-treated structure is almost solely used with an asphalt pavement, /20/. In other countries the use of cement-treated materials is not so widely spread, although interest and test activities are great.

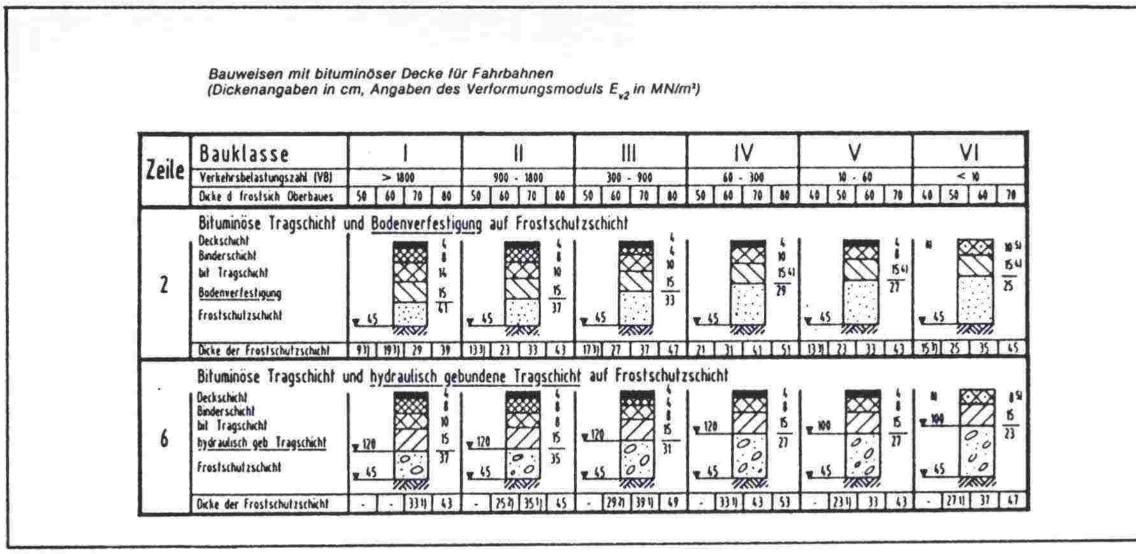


FIGURE A-26. Semi-rigid cement-treated structures in the German standards /30/

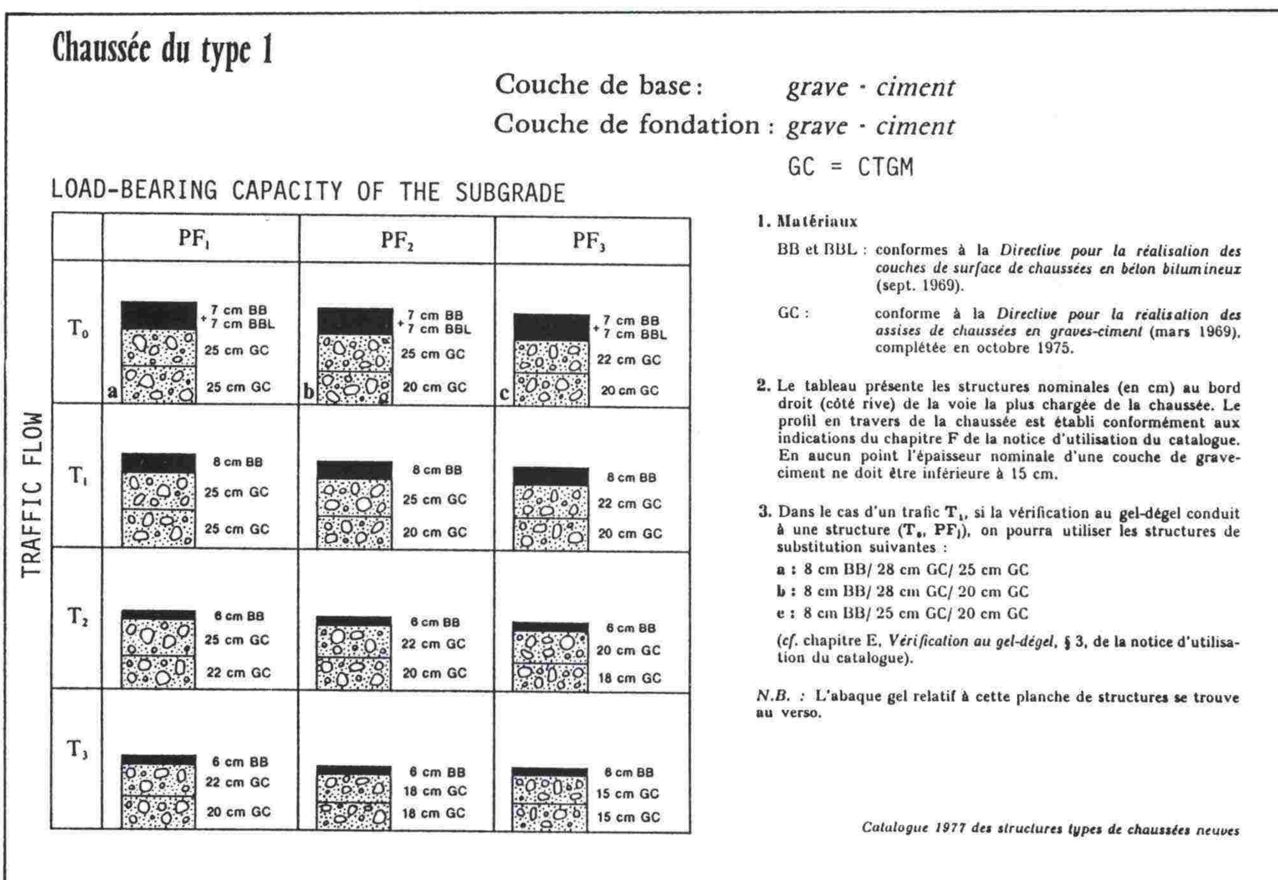


FIGURE A-27. An example of cement-treated structures in the French standards /20/

Schweizerische Normenvereinigung (SNV) Association suisse de normalisation

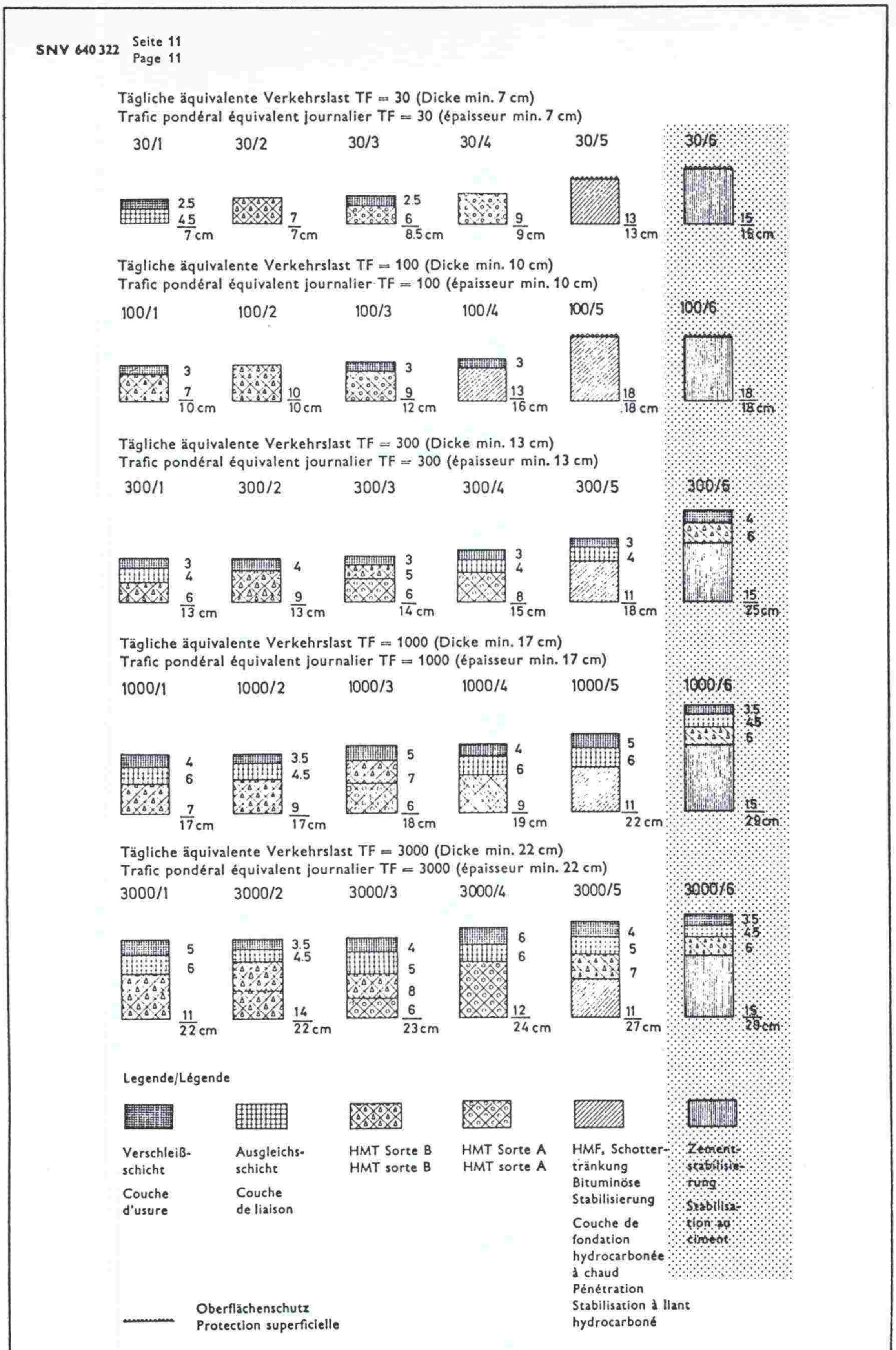


FIGURE A-28. Structural alternatives for the upper part of the pavement structure in the Swiss standards /22/

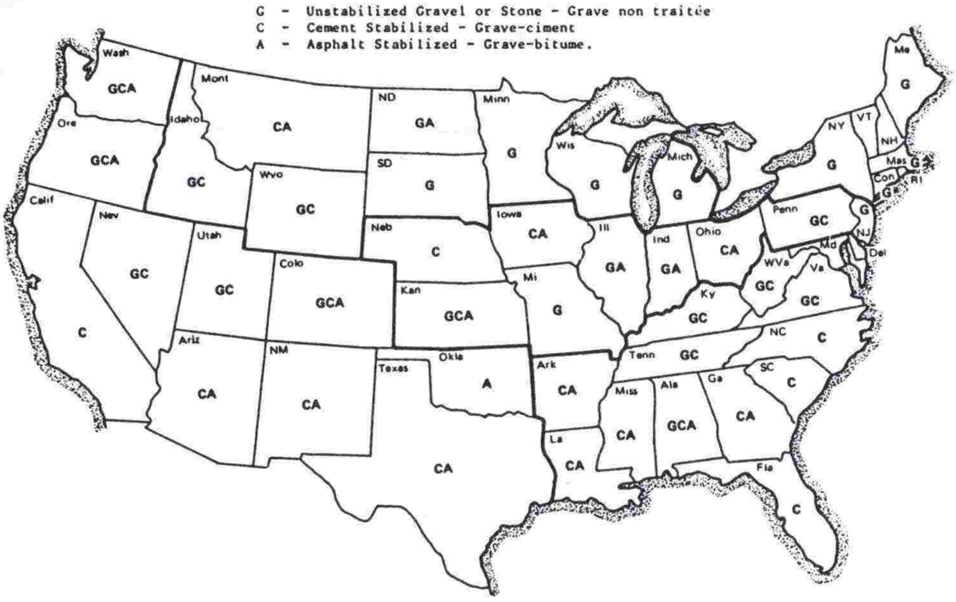


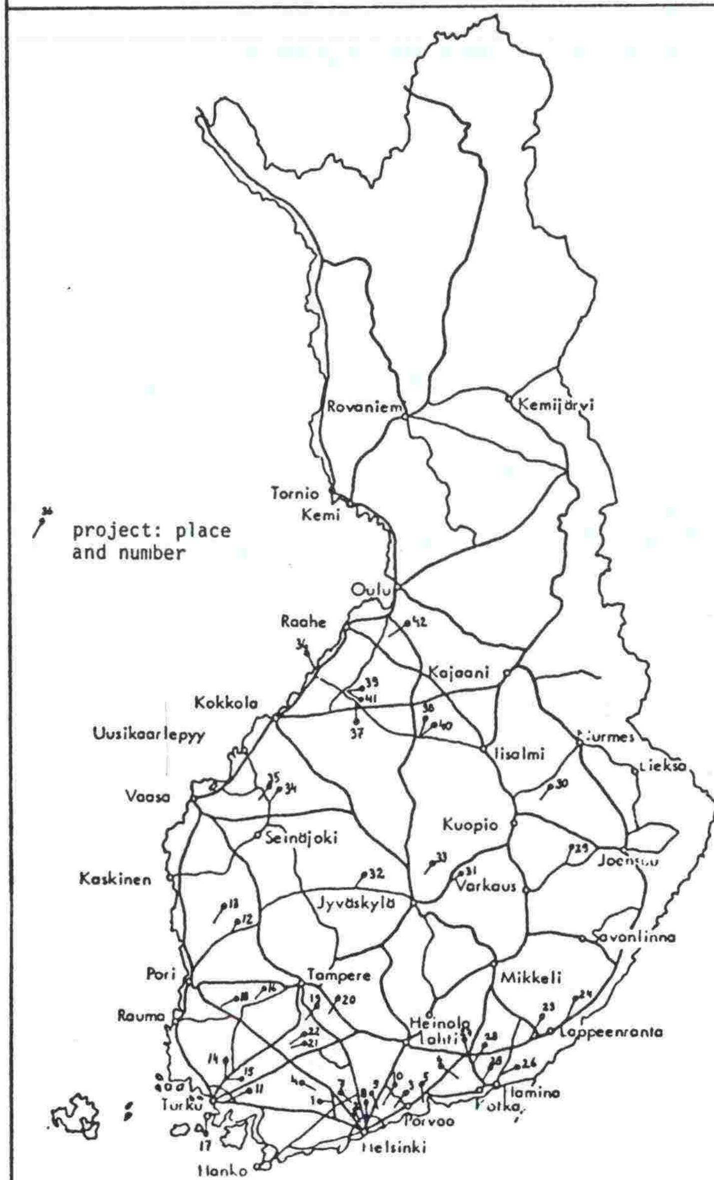
FIGURE A-29. The base structure of concrete pavements in the different states in America

Of the Scandinavian countries Norway has strongly increased the use and research of cement-treated pavements, /10/.

In Denmark cement-treated pavements are always built as a base of a concrete pavement; otherwise interest has been little, although in the norms there are alternatives also for cement-treated pavements in a base course. Also in the Swedish design instructions cement-treated pavements have been mentioned. Because of the thick asphalt layer (100 - 200 mm) required on a cement-treated base or subbase this alternative has not been competitive and it has not been used. New investigation results of using cement-bound macadam together with a thin asphalt layer are encouraging, /36/. To repair old roads cement-treated pavements are not used in Sweden because of the negative experiences in the 1970's.

In other countries the experiences of the durability of cement-treated pavements have mainly been good, especially experiences of the permanence of the load-bearing capacity. Cracking is a 'besetting sin' of a mixed structure and it causes maintenance costs; the costs depend on how sensitively this cracking is taken. Generally a mixed structure is considered as good as a bitumen-bound structure, and it is regarded as less expensive also on roads with heavy traffic.

However, thin asphalt pavements (< 10 - 15 cm) are feared to lead to a greater damaging degree and increasing maintenance costs and that is why the method of thin pavements are under constant investigation. Owing to the negative experiences in Sweden and Canada avoiding of

Roads investigated	Information on roads investigated and on their condition	Notes and recommendations (VTT)
	- 77 stab. works made in 1960-1978, totally 1 457 000 m^2 (ab. 230 km), moreover 130 000 m^2 on airfields - The studies included 42 objects, 90 000 m^2, 73 % of the total amount - 37 % in the base course, 20 % in the base/subbase course (an unbound layer of <10 cm between), 37 % in the subbase course, 1 % in the filter course, 5 % in the subgrade - 2/3 mix-in-place., 1/3 mixed-in-plant - 57 % of asphalt pavements, 43 % oil gravel or cold asphalt pavements - Layer thickness (12-20 cm) 3 % thinner than designed (mixed-in-plant), 26 % thinner than designed (mix-in-place) - Average compr.strength 6,7 MN/m^2 - Bearing capacities equivalent to those in the construction phase. E-modulus based on loading measurements E=1900 MN/m^2 (mix-in-place), E=9700 MN/m^2 (mixed-in-plant) - One transverse crack 54 m, longitudinal cracks 18,4 m/100 m, more single cracks in mixed-in-plant, more alligator cracks in mix-in-place - Initial costs preserved stabile in the course of time	- The mixed-in-plant method gives a more homogeneous and better result. - Equipment for laying and mixing cement is to be developed in the mix-in-place method. - To avoid cracks in the middle the adjoining lanes must be laid within 2 h. - An unbound layer between the pavement and cement-treated material essentially reduces reflection cracking. - Drilling samples should be taken to follow up the strength and thickness of the layer. - A good compaction decisively important, the base must have a sufficent load-bearing capacity. - Stones are a great hindrance in the mix-in-place method. - The economic situation improves when using slag and moraine **FIGURE A-30. Condition of cement-stabilized roads, a summary of the publication of VTT No. 62/1980/37/**

cracking damage and other durability conditions must be carefully considered when cement-treated pavements are designed in cold and frost-susceptible conditions.

A 62 EXPERIENCES FROM FINLAND

A 621 Performance tests of former cement-stabilized roads

Yearly about 100 000 m^2 of cement-treated pavements and cement stabilization have been built on public roads in Finland, little amounts have also been used on streets and yards, Table A-1. A thorough research of the condition of these cement-treated pavements was done in 1979 by the Roads and Waterways Administration and VTT, the Road and Traffic Laboratory, /37/. A corresponding research was done in all Scandinavia. VTT received information on 42 objects, which was equivalent to about 900 000 m^2 of cement-treated pavements. The research covered about 75 % of the total amount. A summary of this research is in Figure A-30.

Contrary to Sweden this research gives a rather positive picture of the durability of cement-treated pavements in the Finnish conditions. Although about a half of the objects were in a base course the inconveniences of cracking were not stressed. In general, cement-treated pavements had prevailed their load-bearing capacity or improved it in the course of years. The deficiencies are often related to the working method or to a small size and vagueness of objects. The recommendations of the researchers for future projects have also been marked on the table.

A 622 Cement-treated pavements on the Palojärvi-Olkkala test road

In the 1970s a considerably wide research programme on road structures was completed in Finland. It concentrated on the test section on the Palojärvi-Olkkala road (later called Main Road 2) in the Vihti commune. This test road was designed as a co-operation between several parties in 1971 - 72 and it was built in 1973. For a follow-up a 5-year research programme was made out and the final report was published in 1979, /8/. In connection with the research work many separate reports were published, among others theoretical calculations of pavement structure alternatives, /9/.

The test road consisted of 27 test sections each being 100-300 m, Figure A-31, seven (7) of them were cement-treated structures paved with asphalt (MB1-MB7). Ten (10) test sections were paved with concrete (B1-B10), and in four of them cement-treated base as a part of the pavement structure (B1, B2, B7, B9) were used. The aim of the research was to study in comparable conditions how different kinds of structures are suited to the Finnish climate and frost-susceptible soil. As to the cement-treated pavements, the durability of different cement-treated structures in a relation to each other and to other comparison objects was investigated. All structures were so designed that frost action was possible; thus many test lengths were intentionally short-dimensioned. When cement-treated pavements were designed a value of 5 - 6 MPa was used as a design strength in a base course, E-modulus was 7000 MN/m^2. Correspondigly in a subbase course the design strength was 3 - 4 MPa and the E-modulus 4500 MN/m^2.

In the follow-up research settlements, frost heave, friction, abrasion, evenness, bearing capacity, damage and driving comfort were measured.

It could be stated in general that those lengths where cement-treated pavements were used were also damaged quicker. They had cracks, breakages and ravellings which required repair actions during the follow-up time. These defects resulted from faults made during the working process on the surface of the cement-treated pavement (ravellings) and from a short-dimensioning that was more apparent in these structures than in the others.

When comparing the cement-treated pavement sections with each other the results support the opinion that even a thin cement-treated layer increases efficiently the load-bearing capacity of a structure, but in order to reach a long life period a sufficient frost protection or thicker cement-treated pavements are required. When using a thin asphalt pavement of 5 cm it results in hair cracking, and the deterioration of a cement-treated surface can be disastrous to the durability of the pavement. Instead, with a pavement of 10 cm a damage risk on a road surface can be avoided.

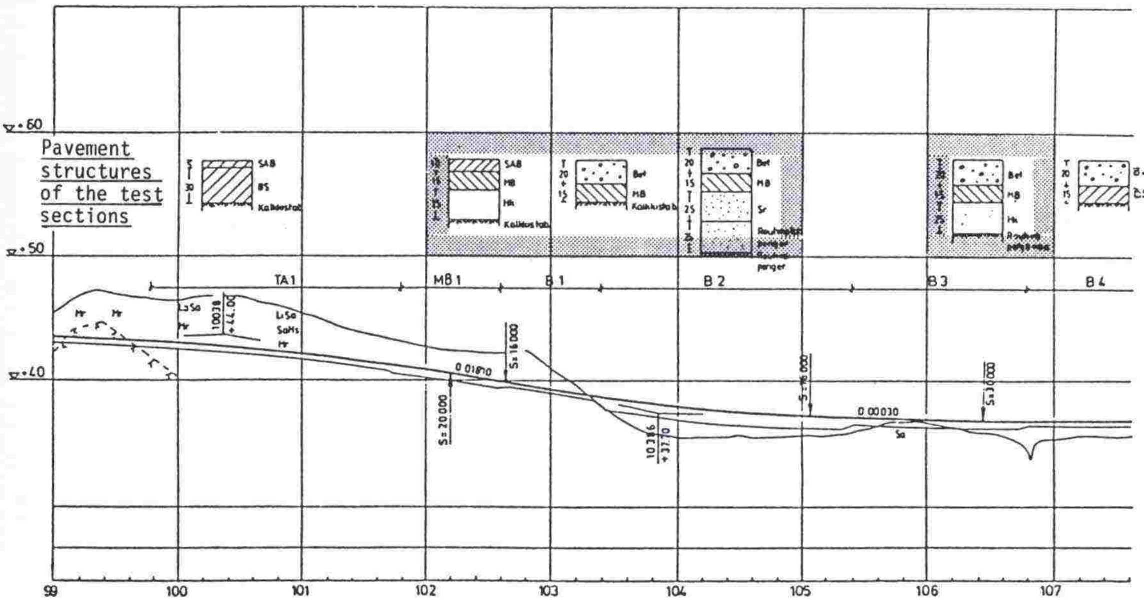
A 623 Other Finnish experiences

Primarily the Roads and Waterways Districts together with cement manufacturers have acquired experiences among other things of the utilization of moraine and blastfurnce slag in cement-treated pavements as well as of working methods, /12, 13, 15/.

In the Finnish soil bed we have plenty of frost-susceptible, stony moraines so that improving their validity with stabilization is in a way a possiblity which has not yet been exploited. In principle it is always possible to stabilize moraine, but the fact that the subgrade is so stony and rich in humus makes it uncertain whether stabilization in practice is profitable and appropriate. Careful preliminary tests are needed when designing stabilization of moraine and cement-treated pavements based on moraines. Best are moraines which are rough and contain a reasonable amount of fines; they can be crushed without the equipment being choked up and then handled. Addition of cement makes them applicable on any pavement structure. Possibilities to use moraine have been studied not only with field tests but also with investigations by the Road and Waterways Administration, research centers and universities, /12, 13/.

Also possibilities and qualifications to use blastfurnace slag have been studied with investigations and field tests. Finland has a positive attitude to blastfurnace slag in road structures. Hence, the use of blastfurnace slag is allowed up to 70 % of the total amount of binders.

In stabilization of old roads the stones in the material to be stabilized has usually been the hindrance. Several methods have been developed to remove stones and to mix stony materials. In the Road and Waterways District in Keski-Pohjanmaa a plough-like mixer has been developed to stabilize rocky materials. Except for rockiness also lack of effective mixers has given a "handmade" impression on in-situ stabilizations and hampered a wider use. In 1987 special equipment for mixing material has been acquired to Finland. The first experiences show that they have a good mixing and working capacity, (Figure A-14). Even these machines are unable to work with very stony materials.


Pavement structures of the test sections
TA1
MB 1
B 1
B 2
B 3
B 4
99
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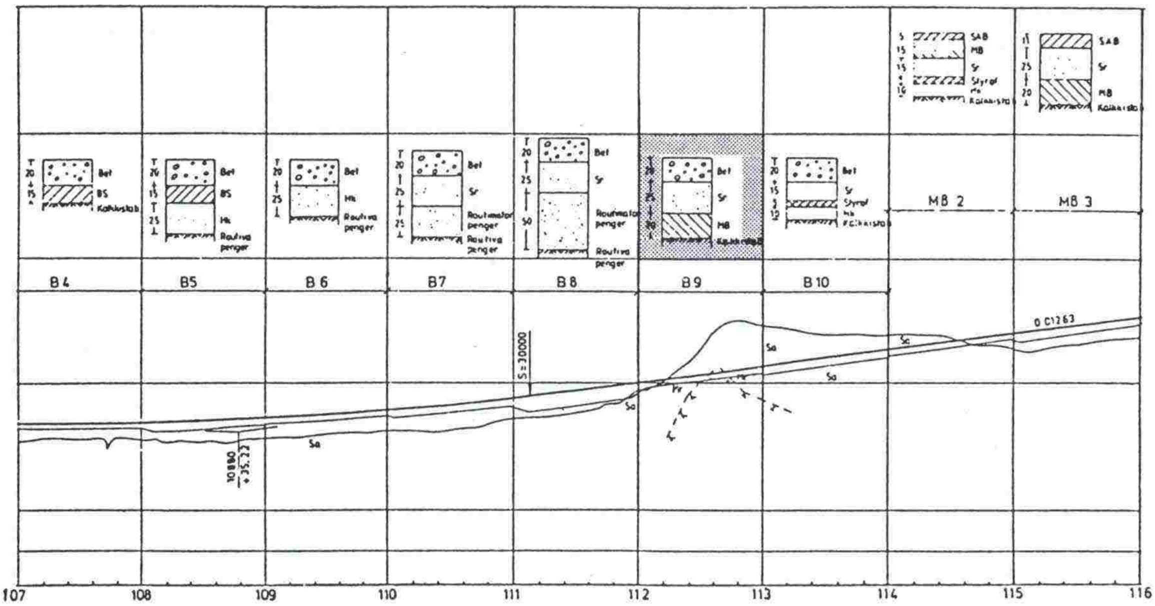
B 4
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B 8
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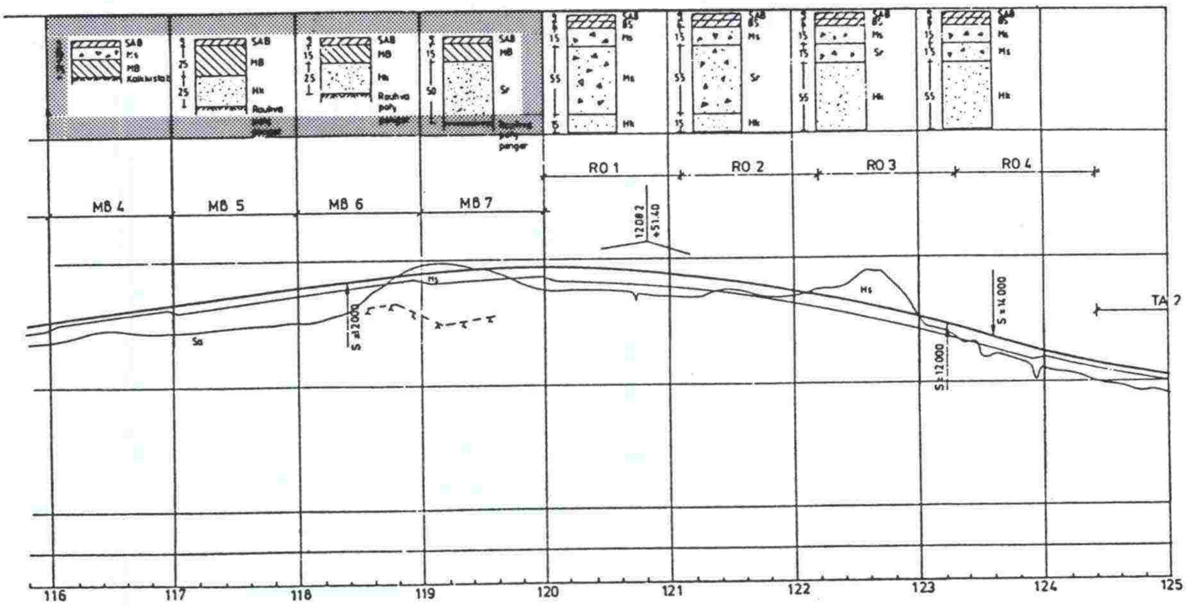
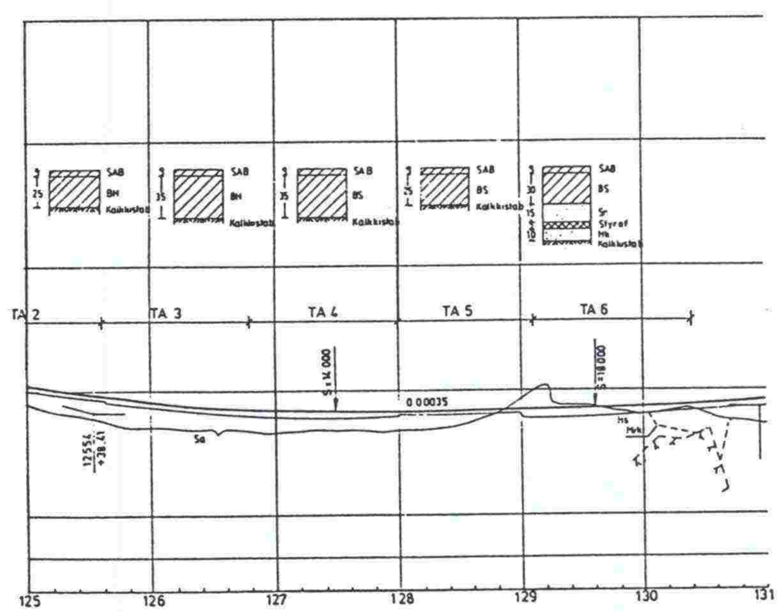




ABBREVIATIONS

FORKORTNINGAR

SAB	Gravel-asphalt concrete Grusasfaltbetong
BS	Bitumen gravel Bitumenstabiliserat grus
BH	Bitumised sand Bitumenstabiliserad sand
Bet	Concrete pavement Betongbeläggning
MB	Cement-treated layer Jordbetong
Sr	Gravel Grus
Hk	Sand Sand
Ms	Crushed gravel Krossgrus
Styrof.	Thermal insulation styrofoam Styrofoam värmeisolering
Kalkkistab.	Lime stabilization Stabilisering med kalk

PALOJÄRVI-OLKKALA
TESTROAD
Longitudinal section

FIGURE A-31. The longitudinal section of the Palojärvi-Olkkala test road and the pavement structures of the test sections

One third of the cement-treated pavements in Finland has been made as a mixed-in-plant process. As to the quality the experiences of this method have been good in those districts where the work has been done. In many cases, however there has not been an easily transferable mixing plant with a sufficient capacity. This has delayed the working progress on a road and raised the price.

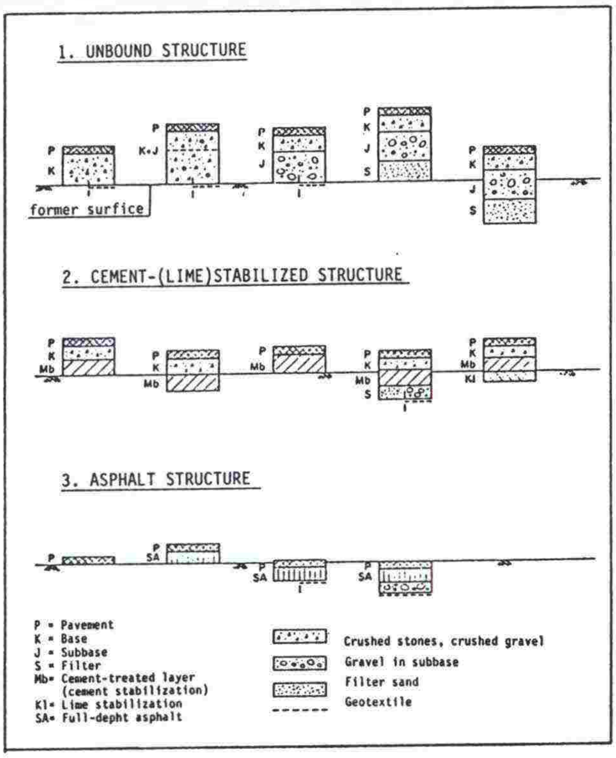
A 7
CEMENT-TREATED PAVEMENTS IN THE FINNISH STANDARDS AND SPECIFICATIONS

In a modern sense cement-treated pavements are known in Finland since 1960, when they were used for the first time in a base course of a new road (main road 6, Taavetti). Since 1964 we have had design standards and specifications, which have been revised in the course of years. The most important standards and specs are listed in Table A-3. Of them the instructions 5 - 9 represent the new valid design practice in the Road and Waterways Administration. Publications from the 1970's contain

TABLE A-3. Design and construction of cement-treated pavements - the most significant Finnish publications

1.	1965	TVH:n Normaalimääräykset ja -ohjeet	Standars for road design by RWA
2.	1970	Maabetoni / Sementtiyhdistyksen käsikirja /3/	Manual for CTGM construction
3.	1972	Stabilointiohjeet, TVH 73 2614 /4/	Technical specifications for stabilization works
4.	1976	Kadunrakennuksen tekniset ohjeet, KTO 76/SKTY /38/	Technical specifications for city streets
5.	1980	Tien rakenteen parantaminen, suunnitteluohje, TVH 722336 /34/	Design manual for structural improvement of roads
6.	1984	Maabetoniurakan urakkaohjelma, TVH 731462	Contract schedule for CTGM works
7.	1984	TVH:n kirje tiepiireille Rt-16/2.3.84 liitteineen x) /25/	A statement by RWA concerning specs for, design and construction of CTGM
8.	1985	TVH:n Normaalimääräykset ja -ohjeet / Tien rakenne /40/	New standards for road design by RWA
9.	1985	Maabetonityön työselitys, TVH 731464/ 21	New specs for CTGM works
10.	1987	Betonipäällysteet ja sementtistabilointi/	Concrete pavements and cement stabilization, translation of a swedish manual
	x)	(liite 3: Näkökohtia maabetonin suunnittelusta sekä työkohtaisen työselityksen ja urakkaohjelman työkohtaisen osan laatimisesta).	

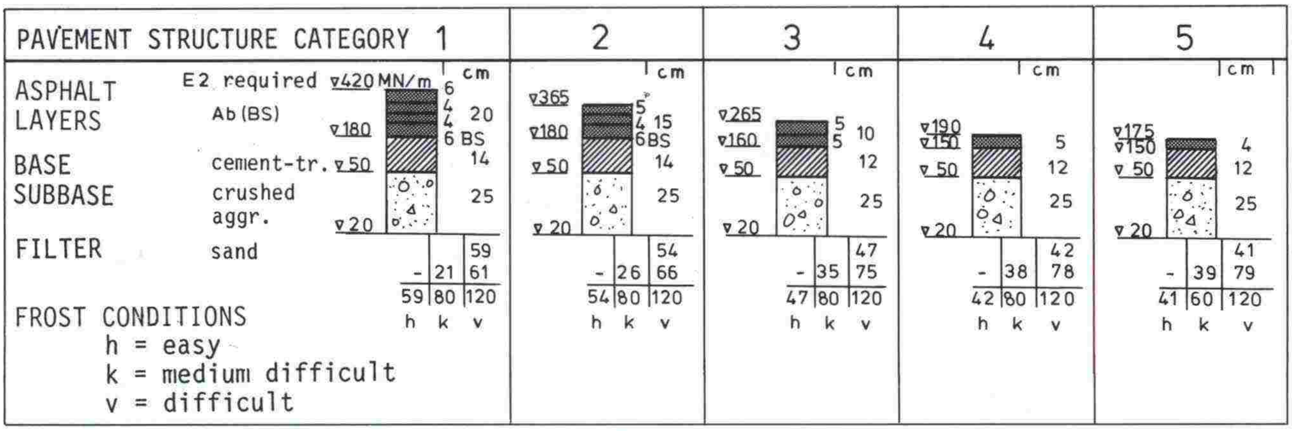
still important and valid basic information, but they have been or will be brought up-to-date. The instruction 10 is a translation from the Swedish publication "Betong på mark" and contains the latest view of the design and construction of cement-treated pavements in Sweden. Figure A-32 presents alternatives for improving the structure of an old road. In Figure A-33 there is an example of cement-treated structures of a new road according to the instruction 8 in Table A-3. In the instructions cement-treated pavements are presented as an alternative for other structural solutions and its design and construction are dealt with in an adequate way. Thus general conditions exist for the use of cement-treated pavements. In the instructions bound base courses have no favoured position in any pavement class, so that cement-treated pavements have to compete in price with other alternatives, although they offer a more rigid and thus more durable alternative.



Source: TVH 722336/1980

FIGURE A-32. Structural alternatives in the improvement of the road structure /34/

STANDARD STRUCTURES OF ASPHALT-PAVED ROADS IN FINLAND
* Base course: CTGM
* Category of subgrade E
* Frost index 25 000 hoC



Source: TVH/Standars IV 5.3 (16.9.1985)

FIGURE A-33. Cement-treated structures in the norms of the Road and Waterways Administration /40/

A 8
PRICE AND PROFITABILITY OF CEMENT-TREATED PAVEMENTS

Direct price

The price of cement-treated pavement consists of the sum of the costs of different working phases, Figure A-34. Compared with an unbound layer the direct additional costs are caused by the supply and mixing of cement with auxiliary works. In Figure A-35 a typical m^2-price and a price distribution for cement-treated pavements in a base course are presented. It can be roughly estimated that stabilization costs of aggregates are two times the cement price in the site.

When comparing a m^2-price of a ready-compacted layer as unbound with a bound layer, the cost difference is 1,5 times the price of the cement itself.

The price of cement in the site (2/88) varies 0,40 - 0,50 mk/kg (1988) depending on the quality and delivery distance. Blast-furnace slag costs about 0,25 mk/kg ex works (Raahe, Lohja) (1988).

The share of aggregates in the price of cement-treated pavement varies between 30 - 50 % depending on the source of aggregates and transportation distance.

Prices in Figure A-35 give only a size range; price of cement-treated pavement must be calculated case by case, /41/.

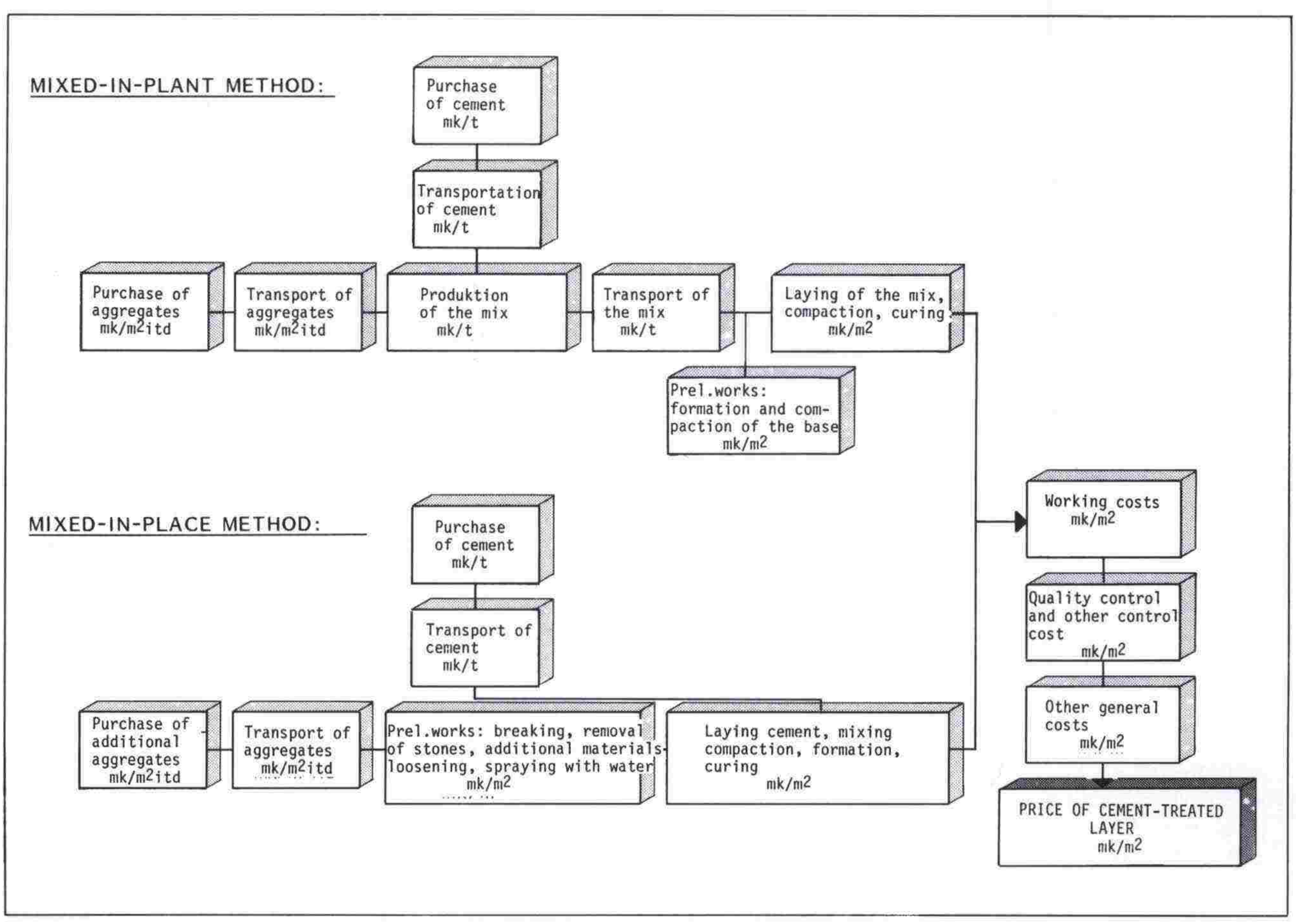


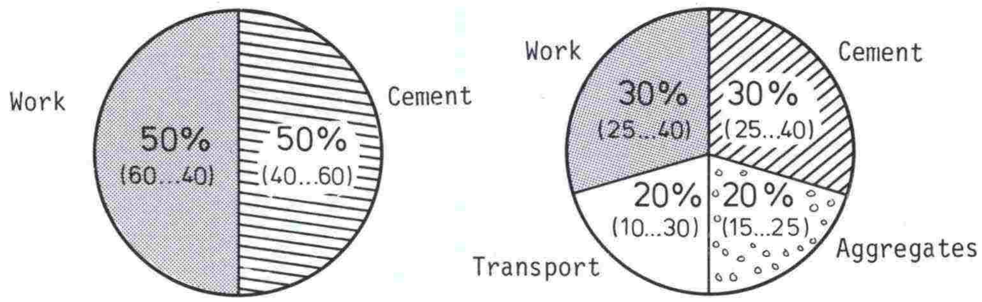
FIGURE A-34. Working phases and cost development of cement-treated pavements

TYPICAL m2-PRICE: (15 cm cement-treated materials in the base course, cement content 5 %, hauling distance 15 km)

Without aggregate	Including aggregate
17 mk/m^2	29 mk/m^2
(14...20)	(22...32)
(incl. joint costs)	(incl. joint costs)

THE CHANGE IN THE PRICE WHEN THICKNESS CHANGES 1,10 mk/m^2/cm

TYPICAL PRICE DISTRIBUTION:



NOTE! The information of the figure gives only the size range based on completed projects. Actual costs of single cement-treated projects depend greatly among other's on the amount of cement, hauling distances, size of the project and working methods.

FIGURE A-35. Price of cement-treated materials in Finland in 1988

FIGURE A-36. Construction costs of cement-treated pavements with different working methods

Example: a cement-treated pavement of 17 cm on the subbase course, area 50.000 m^2 (10 m, 5 km), aggregate crushed gravel 0...45 mm, hauling distance from the crushing plant 30 km, compaction coefficient 0,70. Amount of cement 4,5 % = 17 kg/m^2, 75 kg/m^3, in the mixed-in-plant method the plant on the site (distance to the road 0-3 km), the project in southern Finland

MIX-IN-PLACE METHOD					MIXED-IN-PLANT METHOD			
quantity	unit	mk/unit	mk		quantity	unit	mk/unit	mk
11500	m3itd	17,00	195 500	AGGREGATE IN STOCKPILE	11500	m3itd	17,00	195 500
11500	m3itd	26,40	303 600	LOADING AND TRANSPORT 30 KM	11500	m3itd	26,40	303 600
				LAYING WITH GRADER	50000	m2	0,34	17 000
850	t	370,00	314 500	CEMENT ON SITE	870	t	370,00	321 900
350	h	600,00	210 000	PRODUCTION OF THE MIX (60 t/h, plant + one loader) / STABILIZATION: spraying with water 150 mk/h, scarifying 200 mk/h, laying cement 170 mk/h, levelling 170 mk/h; 690 mk/h	100	h	690,00	69 000
11500	m3itd	5,22	60 030	HAULING TO THE ROAD				
350	h	300,00	105 000	SPEEDING WITH ASPHALT PAVERS (20 000 t mix)				
350	h	190,00	66 500	COMPACTION	100	h	190,00	19 000
50000	m2	1,60	80 000	CURING (0,6 kg/m^2BE)	50000	m2	1,60	80 000
TOTAL WORKING COSTS			1 335 130	⇒ 26,79 mk/m^2 / 20,12 mk/m^2	TOTAL WORKING COSTS			1 006 000
			160 215	JOINT COSTS 12 %				120 720
TOTALLY			1 495 345	MIXED-IN-PLANT 29,91 mk/m^2 / MIX-IN-PLACE 22,53 mk/m^2	TOTALLY			1 126 726

Price of cement-treated pavements with different working methods

Generally it is said that with a mixed-in-plant method a more homogeneous quality can be obtained and the price won't be higher than in the mix-in-place method, if the material to be bound is brought from elsewhere in both cases. In many countries it is not allowed to use the mix-in-place method in the base course and then it is not necessary to compare the prices. However, the mix-in-place method is developing all the time as to equipment and working precision. Thus it is important to follow the quality/price relation of both methods. So far in Finland the mixed-in-plant method has been 20 - 30 % more expensive than the mix-in-place method. The reasons are very natural:

- Worksites are small; transfer costs of a mixing plant are emphasized in the price.
- Easily transferable mixing plant equipment is scarce.
- There are not many continuous mixing plants with sufficient capacity.
- Worksites are opened only occasionally.

The price relation will be more in balance when there is more work and better equipment. The actual costs of aggregates must be taken into account in price calculations. In Figure A-37 a calculation form and a comparison example are presented.

Profitability of cement-treated pavements

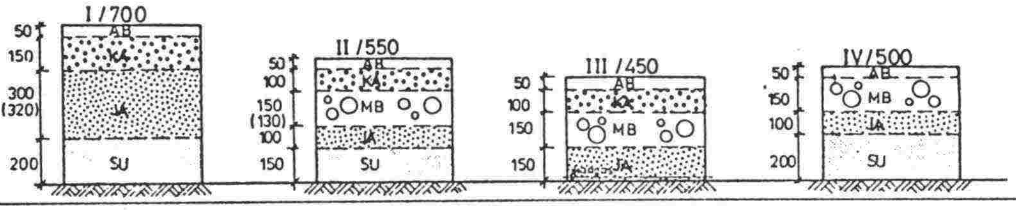
A cement-treated pavement can be part of the standard structure of a high-quality road when its use is based on a road class and traffic amounts and then no profitability calculations are needed.

In general, however, a cement-treated structure is a structural alternative which has to show its competitiveness when cost comparisons are made. When this kind of a comparison is made it should be performed as a total comparison of properly designed pavement structures, Figure A-37.

The savings in transportation costs of aggregates used in cement-treated pavements - but most of all in materials and thicknesses of other pavement layers - compensate the additional costs caused by the addition of cement. When all alternative costs per unit length of the road are calculated a correct starting point for a cost comparison is obtained. When costs of cement-treated pavements turn out to be the same as or only a little more expensive than those of other alternatives, cement-treated pavement should be chosen because its rigid structure secures a longer life period.

The profitable comparison can also be made based on life cycle costs according to chapter B 162. However, there is so little follow-up information on the durability of cement-treated structures and so little experience of its design in Finland that we have to rest content with comparisons of construction costs. These comparisons can be completed with estimations of maintenance costs based on experience. There are many methods for making a cost comparison; Figure A-37 is only an example. Computer programs make the practical calculation work easier; in the Road and Waterways Districts there is an application for Olivetti M 24 available which speeds up the comparison of different structural alternatives.

Category of subgrade E   Pavement structure 4
Initial bear.capac. E_2=20MN/m2   Target bearing capacity under the pavement E_2=150MN/m2



COST COMPARISION OF STRUCTURES

Pavement structure	Road width	Slope fill		Geotextile		Filter		Subbase		Base		Cement-treated		Comparision cost
	m	m3rtr	mk	m2rtr	mk	m3rtr	mk	m3rtr	mk	m3rtr	mk	m2rtr	mk	mk/jm
E/I/700	7.0	0.74				1.76		2.41		1.11				
	8.0					1.96		2.72		1.26				
	9.0					2.16		3.02		1.41				
E/II/550	7.0	0.45				1.26		0.91		0.73		7.00		
	8.0					1.41		1.01		0.83		8.00		
	9.0					1.56		1.11		0.93		9.00		
E/III/450	7.0	0.30		8.35				1.32		0.73		7.00		
	8.0			9.35				1.47		0.83		8.00		
	9.0			10.35				1.62		0.93		9.00		
E/IV/500	7.0	0.38				1.64		0.83				7.00		
	8.0					1.84		0.93				8.00		
	9.0					2.04		1.03				9.00		

UNIT PRICES:

Slope fill	mk/m3 rtr	Cement-treated pavement	(150)	mk/m2 rtr
Geotextile	mk/m2 rtr	—"—	(200)	—"—
Filter	mk/m3 rtr	Pavement	AB	—"—
Subbase	—"—	—"—	ÖS	—"—
Base	—"—			

FIGURE A-37. An example of the cost comparision of different pavement structure alternatives

A 9
NEED AND QUALIFICATIONS FOR THE USE OF CEMENT-TREATED PAVEMENTS IN FINLAND

A 91 NEED

Cement-treated pavements are not much used in Finland. There are many reasons for this:

- A comparatively good supply of first-rate unbound materials
- The amount of traffic has not required the use of bound base courses.
- Lack of equipment, skill and experience to make cement-treated pavements of good quality.
- In the 1960s and 1970s we had bad experiences of the repair work of old gravel roads by stabilizing.
- No technical and economical profitability comparisions have been required.

In the 1980s the load-bearing defiencies in our road network have become evident. The roads once built and improved have been damaged after having served their planning time and now they require structural repairs and increase of the load-bearing capacity. The improvement of the load-bearing capacity of the road network is one of the main points in the Road - 2000 programme of the Road and Waterways Administration. Every year approximately 1500 - 2000 km of road will be repaired and the improvement of the load-bearing capacity is an essential part of the work.

The expected increase of axle loads at the end of 1990 increases the need for design of better load-bearing structures with a longer life period; this concerns both old and new roads.

To achieve the required bearing capacity with unbound structural courses and with thin asphalt layers is in practice more and more difficult and expensive. Furthermore, an unbound structure does not offer a sufficiently good durability in heavy traffic.

The fact that it is getting more and more difficult to obtain ridge materials and also the long transportation distances urge to find new solutions in many parts of the country. For economical and environemental reasons the use of moraine and industrial wastes in road structure will be emphasized in the course of years.

For all these needs cement-treated pavements offer a good and competitive solution when used in a correct manner.

A 92 QUALIFICATIONS

As a road construction material cement-treated pavements are a reliable and durable solution for a routine use as well in Finland as in other countries. The following conditions must be stressed owing to the soil and climate conditions:

- good drainage
- sufficient frost protection and transition wedges
- frost resistance
- a sufficient asphalt thickness on cement-treated layers
- a sufficient load-bearing capacity of the base

Other conditions for a good load-bearing capacity and a long life period are a professional design of cement-treated layers and first-rate performance of the work. Although in principle cement-treated pavements can be built with usual road construction equipment and machinery, only special machinery and equipment guarantee a first-rate quality and competitiveness. We have no machinery and equipment for extensive use of cement-treated pavements in Finland yet, but they are easily available abroad whenever needed.

A 10
FUTURE DEVELOPMENT

Although the design and use of a cement treatment has already been standardized in most countries and although experiences of its durability and behaviour are generally positive, there is still a certain need for further studies:

- As to base courses the cement-treated base should have a minimum asphalt thickness on top of it.
  This sets new demands for aggregates, proportioning, the quality of work etc.

- To be able to control cracking and reflection cracking further studies are required.

- The E-modulus of cement-treated pavements has not been sufficiently studied and this results in a great variety of design principles of the load-bearing capacity.

- Need for recycling of materials leads to the fact that materials with unknown technical qualifications are used as base material of cement-treated pavements, their application must be studied.

- It is a world-wide demand that structural designs of roads must be technically and economically profitable. Thus the product and its qualifications must be thoroughly understood.

- So far cement-treated pavements have been more used in standards than in the field. There is also competition in traditional sectors, asphalts as a base of concrete pavements are an example of this. This competition calls for further studies and development.

The need for the improvement of the load-bearing capacity is so obvious on the Finnish roads, old and new, that it is very likely that also the use of cement-treatment will increase. Then also research and development work must be active. We need investigations and experiences at least of the following points:

- working methods and equipment
- field investigation methods of the quality control
- the minimum thickness of an asphalt pavement
- E-modulus both in the laboratory and field
- load-bearing requirements of a cement-treated base
- costs of different structural alternatives
- the behaviour of different structural alternatives
- the use and durability of cement-treated pavements in the improvement of the load-bearing capacity of old roads.

In Figure A-38 some conceivable structural alternatives and research objects related to them are presented. Although according to chapter A 5 cement-treated pavements can be designed to any structural course, the main interest in Finland is aimed at the development of structural solutions in base and subbase courses.

A 11 SUMMARY

Cement-treated pavements have been used for decades as a base of a concrete pavement and in structural courses of asphalt paved roads. The experiences have been positive. The course of development has been from lower to upper courses up till base courses of high-quality roads. At the same time the traditional picture of the requirements set on the components and load-bearing capacity of cement-treated pavements has changed a lot.

Earlier the stronger demands for the road structure were met with other solutions, for example asphalt or lean-concrete instead of cement-treatment. Now cement-treated pavements are developed to meet even the strongest demands. Thus cement-treatment will no doubt have a wide range of use all over the world. The use of rcc can be seen as a benchmark of this development; the strong progress and good experiences of rcc indicate this.

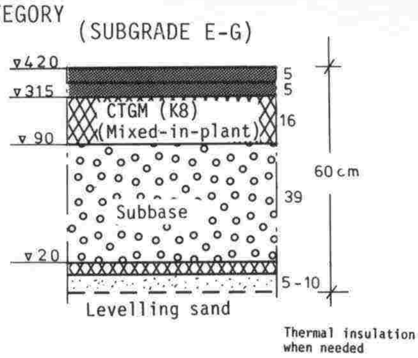
This report is a briefing of the design and use of cement-treated pavements based on both Finnish and foreign literature and on research journeys.

Cement-treated pavements are known all over the world. The design instructions and specifications differ depending on the aggregate and binder materials used. The design practice is also influenced by the attitudes to the cracking of cement-treated pavements. The new cement-treated pavements are more and more often made of first-rate aggregates to base courses and they are designed on the basis of bending tensile stress; the cracking is controlled and the reflection cracking is reduced with surface treatment or a geogrid. The mixed-in-plant

TYPICAL PAVEMENT STRUCTURE CATEGORY

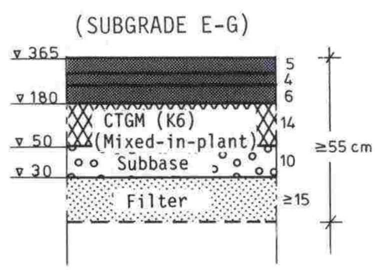
QUESTIONS TO BE INVESTIGATED:

1 and 2



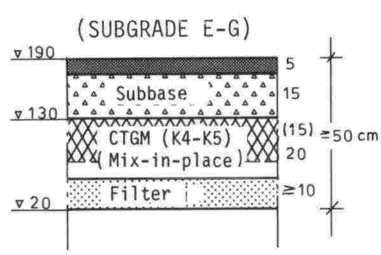
- reflection cracking
- optimumstrength of cement-treated pavement
- the most profitable procedure of thermal insulation
- need for thermal insulation
- frost resistance requirements for cement-treated pavements
- the minimum surface thickness due to the hazard of slipperiness

2 and 3



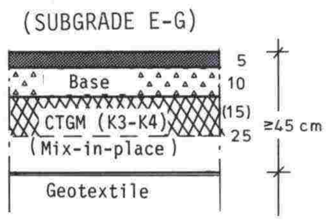
- the most profitable procedure of cement-treated base
- reinforcement requirements for cement-treated pavements

3 and 4



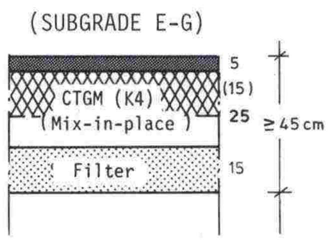
- the most profitable procedure of cement-treated pavements
- the best material of the base course

4, 5 and 6



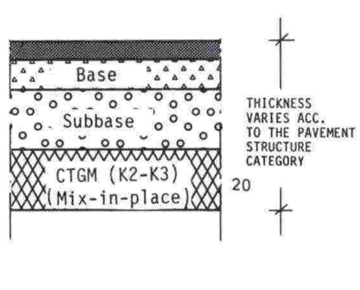
- function of the intermediate layer
- the minimum thickness of the structure

4, 5 and 6



- sensitiveness to frost heave
- the minimum thickness of the structure

1 - 6



- the minimum thickness on the structure
- cost-effectiveness of cement-treatment

FIGURE A-38. Cement-treated structure suggestions and needs for their further development

method has gained ground in base courses, instead the cement-treated pavements on subbases are generally built by using the mix-in-place method.

The amounts of cement-treated pavements vary greatly in different countries and they seem to depend also on the tradition of the road construction in this country. Many states in USA and in Europe, among other things England, France, Belgium and Switzerland have been staunch users.

In Scandinavia the design of cement-treated pavements has followed the practice in Central Europe. The use for the improvement of old gravel roads led to contradictory experiences and stopped its use at the end of the 1970's. In all Scandinavia an investigation of the condition of old cement-treated pavements was made in 1978.

In 1980's Norway has increased the use of bound base courses and in this connection also the use of cement-treated pavements has strongly increased. A thorough study of the durability of cement-treated pavements has also been made in Norway, /10/.

A new handbook has been published in Sweden and cement-treated pavements have been actively marketed. So far, however, only test roads have been built on public roads.

After the middle of the 1970's only some sporadic projects used cement-treated pavements in Finland (about 15 km/year). When the load-bearing deficiencies of the road network were realized in the middle of the 1980's, the interest in cement-treated pavements has strongly increased. Thus in 1987 cement-treated pavements were built about 50 road kilometers.

To improve the load-bearing capacity of the roads with cement-treatment all technical qualifications exist also in the Finnish soil and climate conditions provided that uneven frost action is prevented with a sufficient frost protection.

According to the experiences from abroad the reflection cracking will be at least a cosmetic problem when using thin asphalts and this may cause increasing maintenance costs. In the Finnish experiences the cracking has been regarded as a bearable inconvenience.

The profitability of cement-treated pavements as a structural alternative depends first of all on the availability of aggregates and on transportation distances. Generally it is provided that cement-treated pavements is proved competitive. However, to a greater and greater extent, cement-treated pavements are a firm part of certain standard structures of first-rate roads and then comparisons by single projects are not made. In Finland cement-treated pavements are mentioned in the standards as a structural alternative, but in practice comparisons with cement-treated pavements are made only when there is some specific reason for them.

Cement-treated pavements are under constant development and new experiences are received of its functions in different kinds of structures. Development work is active and there is a great international interest in the cement-treatment. Cement-treated pavements will have a good future.

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61. Stabiloidun moreenin käyttömahdollisuuksista tien päällysrakennekerroksissa, H. Hinkkanen, TVL H-piiri 1985

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CHAPTER B1
GENERAL SURVEY OF CONCRETE PAVEMENTS

CHAPTER B 1

GENERAL SURVEY OF CONCRETE PAVEMENTS

CONTENTS Page

B 1 GENERAL SURVEY OF CONCRETE PAVEMENTS

B 10 INTRODUCTION

The emphasis of the report lies on the special issues related to cement-treated and cement-concrete pavements (Chapters A, B2...B5). In this section of the report only a short general view of the qualifications and use of concrete pavements in different countries will be given; this gives us a starting point to be able to study the special issues. This survey is based on the views presented in connection with this investigation work but it is by no means a complete research. Many manuals are available which can help to deepen the general view /5, 22, 26/.

B 11 HISTORY OF CONCRETE PAVEMENTS

Paving also city streets and roads with concrete started at the same time with the development of new applications of modern concrete after the invention of Portland cement (in 1824). It is known that separate concrete pavement projects were realized in many European countries and in the United States in the last decades of the 19th century; this was still the age of the horse traffic. But a wider interest in concrete pavements was aroused only after the turn of the century when motor vehicles and air tyres were invented (1877). That was the time when the roads were required to have a better load-bearing capacity than gravel and a greater evenness than a stone pavement. Correspondingly, the development of air traffic offered a suitable possibility to study the applicability of concrete as a pavement of airfields.

The first concrete pavements were built by tamping by hand and without reinforcement and joints. From the early years of the 20th century transverse joints were taken into use - first expansion joints then contraction joints. Also the reinforcement of slabs and the use of dowel bars became more general. The reinforcement contributed to longer slab lengths and now lengths of 15 m and even of 30 m were used. At first the design of slabs was made by experience but in 1926 the so called Westergaard's formulae was published and this formulae - even as modified -forms still the basis of most design methods of concrete pavements.

Also the working technique was developed: spreading and tamping by hand were compensated with mechanical tampers and pavers as early as in the 1920s; actual pavers equipped with vibrating screeds were taken into use in the beginning of the 1930s.

Pavement construction activities were moderate in Europe still in the 1910s and 1920s. In the United States, however, about 600 million m^2 of concrete pavements were built as early as in 1925. After the universal depression cars and paved roads gained more and more ground also in Europe in the 1930s. In a decade the number of pavements was multiplied everywhere. Especially in Germany the progress was very strong at the time; by 1942 almost 4000 km of four-lane motorways were built in a period of ten years, 2500 km were under construction - and all of them were paved with concrete /4/.

About 60 km of concrete roads were built even in Finland by the end of the 1930s - mostly principal entry roads in Helsinki and Turku - and this amount has not been exceeded since then./26/

Great road construction projects were started all over the developed world with the boom after the War at the end of the 1950s.

Although concrete pavements had to give way to asphalt in these projects, also the number of concrete pavements increased strongly. At the same time the structure and technique of concrete pavements were constantly developed. This improved the serviceability and durability and maintained the competitiveness also in circumstances where more profitable bitumen was used. Concrete has established its position as a pavement of most heavily trafficked main roads.

The most significant amendment in the concrete pavement technique was the utilization of slip-form pavers in the beginning of the 1970s. This caused also changes in the structure of pavements, such as shortening of slabs, omission of expansion joints and slab reinforcement.

The main road network of the developed countries was chiefly built by the 1980s; the main attention was paid to strengthening of old pavements in the circumstances where traffic was continuously growing. Wide road construction programmes are nowadays effected only in developing countries.

Concrete pavements are one alternative to meet new challenges of heavy traffic and old pavements. Design instructions have been improved and manuals on maintenance and repair have been published. Also economical awareness has been better so that a comparison of prices can reliably be made in the design.

To be exact the above does not concern Finland - nor the rest of Scandinavia for that matter. Here the pre-war tradition of concrete pavements was not so strong that it would have continued after the War. Paving was again started from a zero point. For economical and industrial reasons bitumen pavements were chosen. In the 1960s old concrete pavements were replaced with asphalt pavements after 25 - 30 years of service, only a few concrete pavements were built and mainly as a test, see chapter B 17. Lack of skill, equipment and instructions has hindered wider utilization of concrete pavements and application of the latest paving technology. Instructions for structural design have one-sidedly concerned only flexible pavement structures; they have not guided to a wide use and comparison of different structural types.

When solving problems caused by the increase of heavy traffic, abrasion and long-term durability a growing attention to concrete pavements is paid in the past few years also in countries where they are not traditionally used. This is the case also in Finland.

B 12 QUALIFICATIONS OF CONCRETE PAVEMENTS

A road pavement should be even and undeformed, it should have good friction qualities, a good abrasion resistance and as low a noise level as possible. The best pavement is not the one which has these qualities as new but the one which preserves them unchanged even in demanding traffic and environmental conditions.

The following review studies how these requirements can be met with concrete pavements.

1. the demand: an even pavement

Pavement design is started with a presumption that a better longitudinal evenness (PSR 4,5/5,0) will be reached with a new concrete pavement than with an asphalt pavement (PSR 4,2/5,0); a service level measured as evenness will then be longer preserved with concrete than with asphalt. /16/ Many working methods lead to the good initial evenness. Such are automatic levelling in pavers or the fact that a concrete pavement is not to be compacted. However, a good evenness is very sensitive to working failures and thus a high craftsmanship and correct equipment are absolute requirements to achieve an even final result (see chapter B 15). The fact that evenness is longer preserved on concrete pavements than on asphalt is based on the rigidness of concrete pavements and on their ability - to a certain extent - to level the unevenness formed on the road structure with the passage of time. The evenness can be threatened by movements in the road structure (frost heave, settlements) or by movements of slabs in relation to each other (faulting). For this reason the qualifications of a concrete pavement can fully be utilized only when the structural factors - frost protection, subgrade strengthening, drainage, erosion resistance of the base - are in order.

2. the demand: an undeformed pavement

A concrete pavement does not change in shape in different temperature conditions; deformation caused by heate waves are unknown. As to the load-bearing capacity a concrete pavement is more than just a pavement: it forms the main part of the whole road structure. Thus deformation cannot occur even due to a weak load-bearing capacity of lower courses as is sometimes the case with thin asphalt structures.

3. the demand: good friction qualities

Generally you can state that good and durable friction qualities can be obtained with concrete pavements. An even concrete surface would be slippery under motor traffic when wet; for that reason concrete pavements are regularly textured in connection with the construction work - generally by transverse brushing or grooving. In addition to the type of texturing also the grain size of aggregates and the stone type contribute to preserving the friction qualities. When using rough calcareous aggregates traffic may cause polishing of a pavement and then the friction qualities must be restored by transverse milling or by surface treatment. Polishing is no problem in the Finnish conditions, but here the studded tyre traffic wears away the transverse roughening grooves and thus changes the friction qualities of pavement. In friction measurements of worn-out pavements in Finland somewhat lower friction values have been measured for concrete roads than for asphalt pavements, /21/, although the friction values of both road types are sufficient - partly owing to studded tyres.

The roughness (micro and macro roughness) of a concrete pavement plays an important part not only in friction qualities but also in the noise level of the pavement. Growing requirements of the past years have led to wide research and development of the roughness problem and this may mean changes in traditional points of view in the future.

4. the demand: the lowest possible noise level

According to a public view the noise level of both asphalt and concrete pavements is too high. A decrease in the noise level is claimed to be possible by means of pavement design. It is evident, that this kind of development is taking place in pavements (open-textured asphalt concretes, porous concretes, adjustment of micro and macro roughness). In practice concrete pavements are considered to cause more noise than asphalt at higher speeds /27/, although in noise level measurements the differences between asphalt and concrete pavements are barely noticeable. At lower speeds noise level differences are not experienced and cannot be measured./22/

Other qualifications

a) A concrete pavement has a good light reflection capacity which improves safety in traffic and reduces lighting costs compared with darker asphalt pavements.

b) The structural durability of concrete pavements is good also in demanding traffic conditions. Concrete pavement is very appropriate for heavy traffic conditions and if a pavement slab is correctly designed it withstands an almost unlimited flow of design axles.

However, being rigid a slab is sensitive to overloads; even a minor excess of a design load can break a slab and result in cracking. Even when a concrete pavement has cracked and worn out its load-bearing capacity may be good and the old pavement can be used as a base in the construction of a new pavement.

c) The modern paving concrete is a durable material even in a severe climate. A reliable salt-scaling frost resistance of concrete can be obtained with a proper mix design and air entrainment and even heavy salting in winter maintenance does not endanger the durability of concrete according to Central European experiences. Heavy weathering of a concrete pavement (D-cracking, see chapter B 21) has been a problem in some countries but the Finnish stone types are not sensitive to this phenomenon.

B 13 THE STRUCTURE OF CONCRETE PAVEMENTS

B 131 Concrete pavement types

Concrete pavements can be divided into the following main types:

a) Reinforced concrete pavement

In this type of a pavement a steel mesh is embedded in lane-sized slabs of 7-15 m at the depth of 5-7 cm. The mesh is placed either with supports before the casting or by vibrating it to the fresh mix in connection with the casting phase. The task of the steel mesh is to keep hair-cracks caused by temperature variations as small as possible. Because slabs are relatively long, generally at least each third joint must be made as an expansion joint. Dowel bars are used in joints. A reinforced slab has been the basic type of a concrete pavement and it has been widely used. Inconvenient expansion joints, which also weaken driving comfort, and corrosion of the reinforcement nets accelerated by salting have reduced its use. Instead, other types of concrete pavements have gained ground.

b) Continuously reinforced concrete pavements

In this type of paving a longitudinal continous reinforcement (0,5 -0,7 % of the transverse cross-section of the slab) is placed at the middle height of the slab and no transverse joints are made. The reinforcement reduces the tensile stress on concrete. Transverse hair-cracks can be freely formed on the slab (at distances of 1 - 2 m). Inspite of its thinner thickness (about 20 % thinner than other types) a continuously reinforced pavement is in most cases considerably more expensive than other pavement types.

Pavements built on a compressible base and white topping of old concrete pavements are proper objectives of this kind of pavement. Avoiding maintenance of joints is regarded as an advantage of this method, but corrosion of steel and worsening of random cracks with time can be considered a risk.

A continuously reinforced concrete pavement is the main type of a pavement in certain countries - for example in Belgium where the steel industry is strong.

c) Prestressed concrete pavements

In special cases it is possible to equip a pavement with a reinforcement which is prestressed before the casting of the slab (no cables) or with cables after the casting. By prestressing it is possible to obtain slabs without joints the length of which is tens of meters. These kinds of slabs are made for taxiways of bigger airfields or for aprons where point loads are exceptionally high.

d) Jointed, unreinforced concrete pavements without dowel bars

When slip-form pavers and short slabs equipped with contraction joints became general it was considered possible to abandon dowel bars in transverse joints. Load transfer from one slab to another is considered to take place by means of aggregate interlock in a sawn joint. Dowel bars are used only in longitudinal joints. Cement-treated or asphalt layers form the base. In certain countries, as in France, this type of a concrete pavement is almost solely used. Easy working methods and a more reasonable price are its advantages but faulting of joints is a risk in the long run.

e) Jointed unreinforced concrete pavements equipped with dowel bars

To ensure the load transfer dowel bars will be installed in a short-slab-pavement to be built with slip-form pavers. This type of a pavement has become more and more general and it is obviously the most suitable type in the Finnish conditions. <u>Hence, later in this report concrete pavement refers to this kind of a pavement.</u>

f) Roller compacted concrete

Damp (0-slump) concrete mix is spread and compacted in the same way as an asphalt pavement. It is important to the final result that both spreading and compacting machinery has a great compacting capacity.

No reinforcement is used. Joints are sawn as is the practice in short-slab-pavements. As good a structural durability can be obtained with rcc as with traditional concrete pavements.

Rcc is the latest achievement in the concrete pavement technique. Its use as a pavement for city streets and industrial yards is becoming more and more general and tests with high speed roads are made in many countries /39/.

g) Rigid composite pavements

Rigid composite pavements covered with a thin asphalt layer (40-100 mm) can be mentioned as a special case. The concrete pavement type can be any of the above mentioned types. An asphalt layer built on the pavement usually reduces the thickness of a concrete slab.

Part of the objects mentioned in Chapter B 12 cannot be reached with this method but certain advantages are also gained:

- less accurate working precision
- shorter traffic interruptions during the reconstruction phase
- less sensitive to depressions

These kinds of structural alternatives are mentioned at least in the English, Spanish and Ontarian structural standards.

B 132 The structure of pavement slabs and joints

A typical structure of an unreinforced concrete pavement is presented in Figure B1-1. All the joints are contraction joints where the movement of slabs is not more than 1 mm. Expansion joints are made only at firm structures like bridges.

Dowel bars ensure that the position of slabs in relation to each other does not change. A deeper saw-cut in the joint will serve as a crack inducer. Sawing is undertaken as soon as possible after the casting phase thanks to which shrinkage cracking will be steered at the joints. To fill the joint another lower saw-cut is made in order to form a joint spacing of at least 10 mm. The use of hot sealing mixes makes the installation of a joint strip at the bottom of the spacing necessary to avoid penetration of the joint sealant deeper to the joint spacing. The joint sealant should stick hard to the walls of the joint spacing to prevent access of water to the joint in all temperature and weather conditions. At the same time access of sand and other foreign materials is prevented. A joint can also be left unfilled which makes the second saw-cut unnecessary. However, the use of filled joints is more general especially in so called cold countries where freezing of water and impure materials causes spalling at joints.

Slabs used to be just as broad as the lane. In new pavements the outermost slabs often stretch 0,2 - 1,0 m over the lane and are there connected to an asphalt or a concrete shoulder.

The slab length of unreinforced pavements must not be more than 25 times the slab thickness in order to keep the joint movements small. In the example in Figure B1-1 the slab length is 5,0-5,5 m and the joints are right-angled against the centre line. New pavements are built also with skew joints (6/1 with regard to the centre line) and with random slab lengths (e.g. 3,5 - 4,0 - 5,5 - 6,0). This kind of a structure ensures that heavy traffic cannot cause regular dynamic blows against the joints, which would damage them and reduce evenness. Skew joints have proved to be good which makes them popular on new pavements in many countries. However, the use of dowel bars in skew joints causes installing and designing problems and this design has not become general in cold countries.

B 14 DESIGN PRINCIPLES OF CONCRETE PAVEMENTS

B 141 Design thickness of slabs

The load-bearing capacity of a base is an important factor in the slab design, because slabs must be able to resist flexural stresses in the pavement concrete caused by traffic and temperature variations. The main variables in the thickness design are thus the amount of heavy traffic, the flexural strength of concrete and the load-bearing capacity of the base. Also temperature stresses play an important part in the Finnish conditions. The thickness is also influenced by many other factors such as the use of dowel bars, the width of a slab or estimations of pavement abrasion. In the course of decades many calculation methods

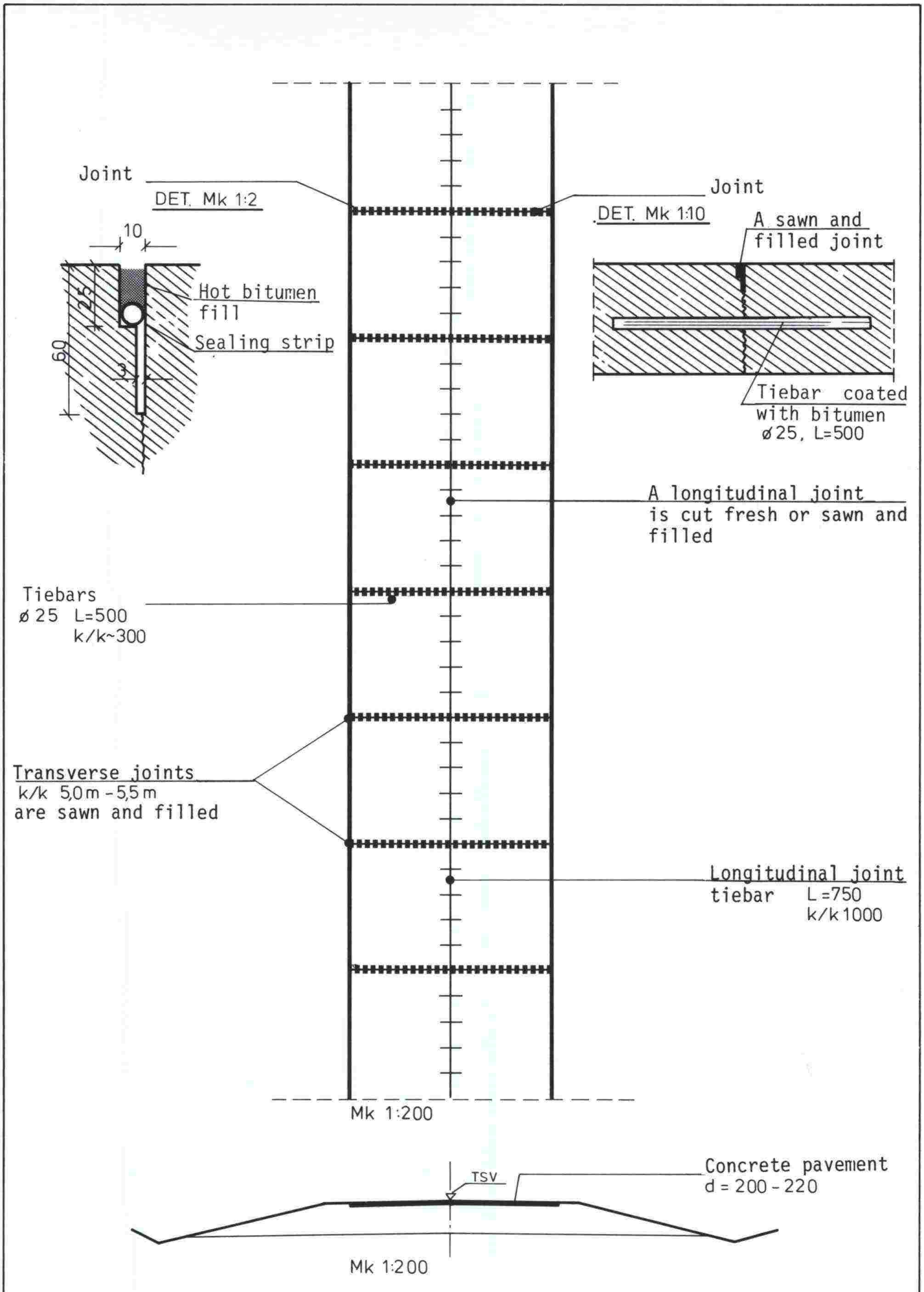


FIGURE B1-1. An unreinforced concrete pavement equipped with dowel bars, a principal structure to design concrete pavement slabs by the aid of these variables have been developed and used. The first and best known of them is the method published by H M Westergaard in 1926; this method is as modified still in use. Another widely-spread method is the AASHO design method developed in the United States, the basis of which are the empirical observations and measurements obtained of road tests. A modified Westergaard method is used in Finland and it is explained among others in the design instructions of concrete pavements (Suomen Betoniyhdistys 1987,/24/). A design diagramme for a slab thickness according to these instructions is shown in Figure B1-2. The diagramme shows among others that an increase in the load-bearing capacity of the base has only a minor influence on the slab thickness. Instead the flexural strength of concrete is an important variable in the thickness design.

The design diagramme and the calculation formulae of the AASHO method is shown in Figure B1-3. Also this method has been further developed and many states in USA and elsewhere have their own modifications of the AASHO method.

B 142 Requirements for concrete

The traditional pavement concrete has been made of sand, crushed gravel (the max. grain size 40-60 mm) and of Portland cement by proportioning them without additives into concrete of quality class K25 - K30. The use of reinforcement and the working methods (side forms, vibrating bars) have, on one hand, allowed and, on the other hand, demanded a high water-cement ratio.

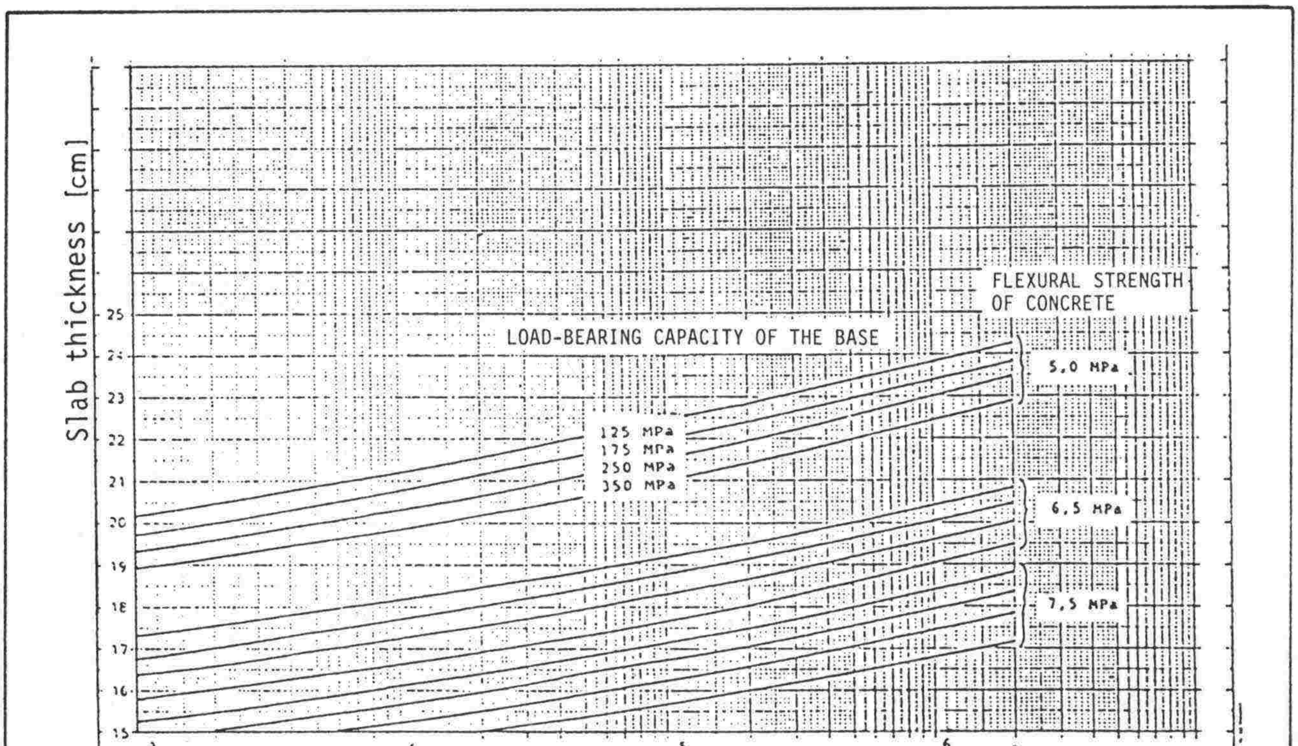
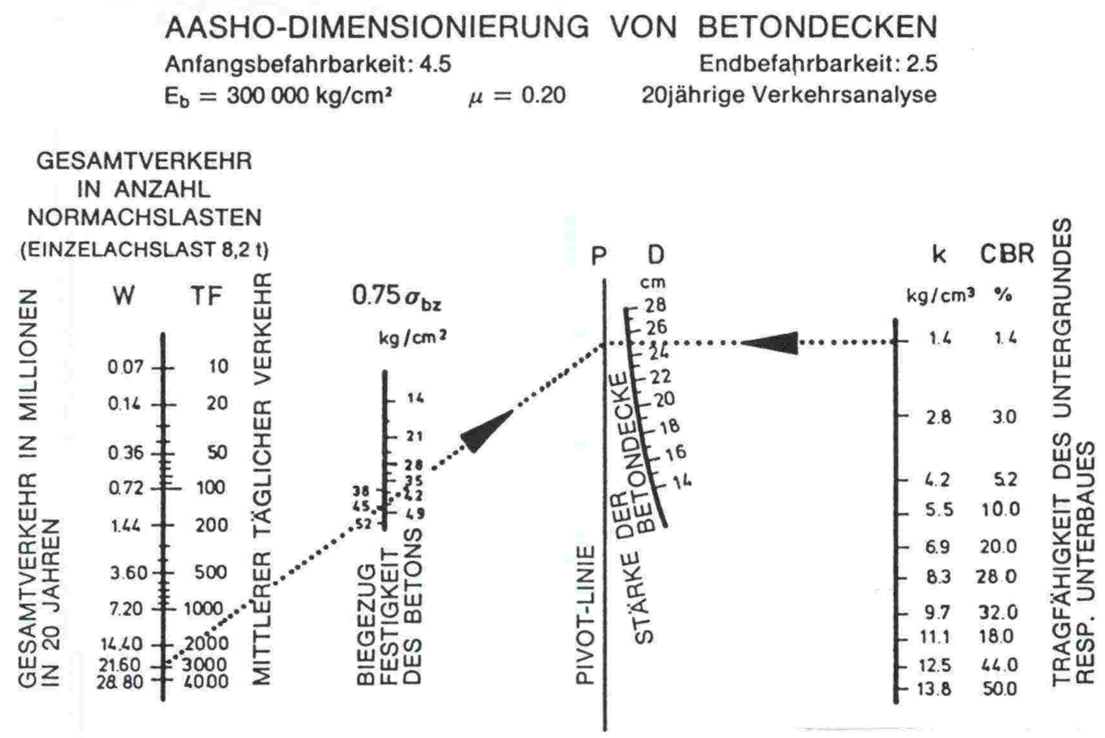


FIGURE B1-2. Design diagramme of concrete pavements; the slab thickness according to the design instructions /24/



$$\log W = \log(7300\ TF) = 7.35\ \log\left(\frac{D}{2.54}+1\right) - 0.06 - \left[\frac{0.176}{1+\dfrac{1624\cdot 10^7}{\left(\dfrac{D}{2.54}+1\right)^{8.46}}}\right] + 3.42\ \log\left[\frac{\sigma_{bz}}{48.52}\left(\frac{\left(\dfrac{D}{2.54}\right)^{0.75}-1.132}{\left(\dfrac{D}{2.45}\right)^{0.75}-23.2\left(\dfrac{k}{E_b}\right)^{0.25}}\right)\right]$$

FIGURE B1-3. Thickness design of concrete pavements according to the AASHO method /34/

A decisive factor in the latest development of concrete pavements has been the development of concrete technology and the use of additives. The use of air entraining agents has been significant in meeting the requirements for salt-scaling frost resistance. The use of plasticizers and super-plasticizers made the utilization of efficient and economical slip-form pavers possible. Thanks to additives it has become possible to work with materials that are richer-in-aggregate and also dryer than before and this has improved the flexural strength of concrete.

Pure Portland cement is still most generally used as a binder, but the positive effects of flyash and blastfurnace slag on strength development and workability have been demonstrated in many investigations and their use together with cement is becoming general. In the Finnish concrete for pavements cement and blastfurnace slag are used approximately in equal proportion.

There are still two kinds of attitudes to the use of flyash and slags in road concrete, there are pros and cons. In Finland it is not allowed to use flyash. The doubts are directed at the long-term salt-scaling frost resistance.

Aggregate development has progressed to the extent that more attention is paid to the durability and wear resistance of rough aggregates. The use of crushed rock as a rough aggregate is becoming more general, which improves the load-transfer in joints. The great maximum size of grains is still typical but its significance is apparently decreasing as the quality of concrete is further raised.

The most evident characteristic of road concrete is as good a flexural strength as possible according to Chapter B 141. It is generally considered that compressive strength and flexural strength are in direct relation to each other. This relation is unexceptionally good with road concrete - the flexural strength can be as much as 15 % of the compressive strength - because road concrete is a material rich-in aggregate and its water-cement ratio is small. According to the Finnish requirements the value of flexural strength should be at least 5 MN/m^2, which is relatively easily attainable.

A strength level of K50...K60 (91 days, strength) is the requirement for the compressive strength in Finland; this level may be raised to about K70.

The wear resistance of concrete sets special demands on the Finnish concrete for pavements. These demands and the mix of modern concrete will be handled more in detail in Chapter B 331. The required qualifications for a Finnish concrete pavement are shown in Figure B1-4.

BINDER:	Normally hardening Portland cement P40/28, (no fly ash) and fresh, powdered granulated blast-furnace slag (grading degree ab. 350 m^2/kg). The proportioning is defined with advance tests.
AGGREGATE:	A frost-resistant, rigid, compact aggregate which meets the demands of class I according to the grading instructions of aggregates and those of class A according to the specifications of pavement works (shape index according to the demand of class II). Properties to be investigated with advance tests.
ADDITIVES:	Compatibility of entraining agents and plasticizers is to be shown with advance tests which are carried out with constituents used in the actual work.
CONCRETE:	- flexural strength 7,0 $MN/^2$ - compressive strength 55 MN/m^2 - change in volume in the salt-scaling frost resistance test ≤ 3 % after 50 freezing-thawing cycles - good wear resistance - air content 2-4 % - water/binder ratio 0,37-0,42 - binder amount < 350 kg/m^3 - maximum grain size of the aggregate 32 mm - percent passing of the sieve of 8 mm about 35 % - gap grading of the aggregate (no grains of 4-8 mm) - at least 50 % of the rough aggregates (> 8 mm) crushed - amount of fines (< 0,25 mm) in the mix ≤ 450 kg/m^3

FIGURE B1-4. Main Road 4, Main Road 8 Kempele-Kiviniemi, Oulu, Kempele (1990). Demands for concrete /29/

B 143 Requirements for the base

A general assumption has been that a concrete pavement is independant of the base quality. As far as the load-bearing capacity is concerned the concrete pavement forms the entire road structure and thus the base quality or the load-bearing capacity has no essential meaning in the thickness design. On the other hand, the base quality plays a significant role as regards the long-term durability, which fact is increasingly emphasized in the design. The significance of the base to concrete pavements is emphasized by at least the following views:

- A slab of uniform thickness does not sufficiently even out the load-bearing differences of the subgrade: in the long run the load-bearing differences cause cracking and deformation on the slab, unless they are evened out by lower layers.

- A blameless behaviour of joints in unreinforced concrete pavements (especially if dowel bars are not used) requires good durability of the base.

- Erosion and bad drainage of the base have been the main reasons for a reduced service level of old concrete-paved roads especially in the United States.

- Good compaction, a good load-bearing capacity and evenness of the working base are essential for slip-form pavers, otherwise a blameless evenness of a pavement cannot be reached.

Thus strict demands are set on a concrete-paved base:

1) A sufficient load-bearing capacity. A minimum demand is placed on the load-bearing capacity. The load-bearing capacity of E_2 = 120...125 MN/m^2 on the subbase course is required in Finland and Germany.

2) A good erosion resistance. The heavier the traffic and the greater the rainfall the higher the erosion resistance of the base must be.

3) Good drainage. Rain and melting waters should be steered away from the structure as soon as possible. Joints and bases should not be water saturated in any circumstances.

4) A uniform base quality. It is important for the support of slab edges and joints that the base is of uniform quality. Also slip-form pavers require compactness and uniform quality of the base surface. The base should not absorb moisture irregularly from a fresh pavement. For the sake of equal quality a uniform sub-base course is required as a base in Finland, although it would not be necessary for the load-bearing capacity.

5) Evenness of the surface. In the slip-form method unevenness of the base reduces evenness of the final surface. Especially the evenness of wheel tracks of the paver is important.

These requirements can be met in many different ways. In principle there are three different structural alternatives: an unbound base, a base bound with cement or a base bound with bitumen. The applicability and advantages of these alternatives are dealt with in Figure B1-5. Generally it can be stated that binding of a base is becoming more popular and that an asphalt base is favoured when the surface slab requirements are high.

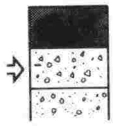
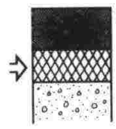
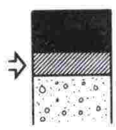
Base quality	Structural principle	Advantages	Disadvantages	Applications	Notes
A. UNBOUND BASE	 Concrete pavement Grushed Gravel Sand	A. - the least expensive way - easy to form - natural drainage	A. - risk of erosion at joints and edges - risk of deformation under a broken, old pavement - slip-form pavers may 'dig down' - an uneven load-bearing capacity	A. - always on inferior roads and with small traffic flows - also with great traffic flows when using continuously reinforced slabs and fixed-form equipment	A. - good permeability important - rough and bearing aggregate important - an unbound base is sufficient as to the theoretical design of the slab
B. CEMENT-BOUND BASE	 Concrete pavement Soil-cement or lean concr. Sand	B. - gives a good working base for the machinery - improves the function of joints - ensures an even load-bearing capacity - reduces stresses on the slab which can be taken into account in the design - use of weaker aggregates and recycled materials possible - evens out effects of frost action	B. - risk of cracks on the slab - compatibility of the base and slab may be difficult - risk of erosion if the cement content is low - it is difficult to achieve a sufficient homogeneity and evenness without slip-form pavers	B. - usually always when the slab is not equipped with dowel bars - more and more often in slip-form projects on main roads - more and more often in all concrete projects	B. - the joint effect of the slab and base arouses much discussion among researchers - in lean concrete aggregate is better, more cement, slip-form pavers - drainage important
C. BITUMEN-BOUND BASE	 Concrete pavement Bit.stab. or asph. concr. Sand	C. - gives machinery the best working base - evenness and homogeneity easily attainable - makes traffic arrangement during working hours easier - supports the slab well	C. - the most expensive construction procedure ? - risk of erosion when using bitumen stabilization	C. - when building by phases - in rehabilitation of old asphalt-paved roads - the use is increasing also on new roads	C. - co-operation between the slab and bit.stab. base is not known

FIGURE B1-5. Structural alternatives of concrete pavements

The following traffic flow values for binding a base are defined in Central Europe/35/ (severe climate, a pavement equipped with dowel bars):

Heavy traffic of a slow lane trucks/24 hours	Base quality
0 - 750	unbound
750 - 2000	soil-cement
2000 - 5000	lean-concrete or asphalt

These values are only a recommendation; the structure of the base must be solved case by case. Also bad experiences of cement-treated and asphalt bases in cold regions have been received in the United States. Open unbound material as a base, efficient drainage with subsurface draines and an abundant pavement thickness are recommended there.

B 144 Structural examples

Standard cross-sections of a new Finnish (Figure B1-6) and of a typical German (Figure B1-7) concrete road are presented as examples of the concrete pavement design. The most significant differences are the concrete pavement on the shoulder in the German and the thick frost protection in the Finnish solution.

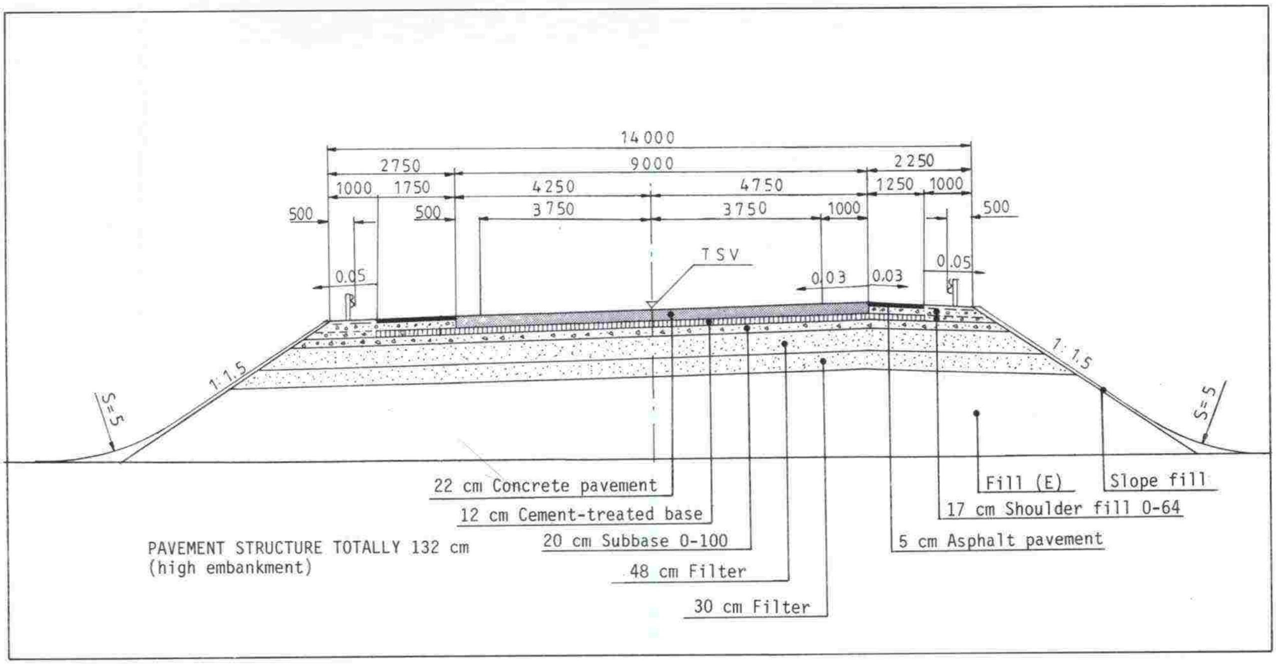


FIGURE B1-6. Main Road 4, Main Road 8 Kempele-Kiviniemi, standard cross-section

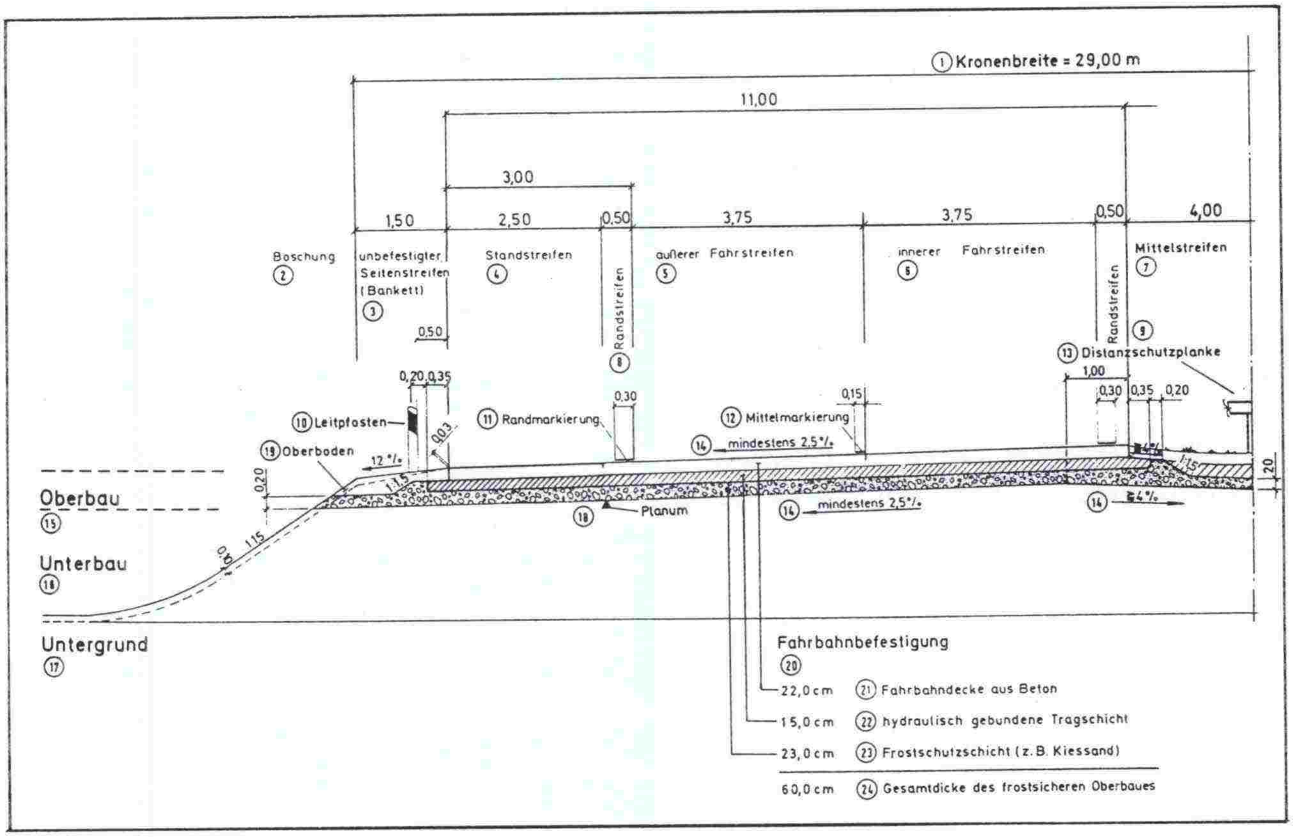


FIGURE B1-7. A typical cross-section of a concrete road in Germany /4/

B 15 CONSTRUCTION OF CONCRETE PAVEMENTS

B 151 Construction requirements

Good evenness and long durability are expected of the concrete pavement. Typical of the rigid pavement is that a successful construction work is of vital importance; afterwards it is difficult or even impossible to restore a service level lost due to a working failure.

Investigations on old concrete pavements reveal that most (60-70%) of the pavement damage results from working failures. This shows that building a conrete pavement is both a demanding and difficult job. In addition to proper machinery and equipment also high professional skill is demanded for a succesful final result. The entire construction process with every detail must aim at the same goal - an even and durable pavement.

B 152 Construction methods

B 1521 The fixed-form method

Typical features of the fixed-form method are side forms and rails, on which the machinery of the different working phases moves, Figure B1-8. Several, simple machines and equipment are used for spreading, compacting and finishing concrete, installing dowel bars, forming joints and curing the surface. The method is the basis of the concrete pavement construction and it was developed as early as in the 1920s. The advantages of the fixed-form method are a good quality level and suitability for construction of different kinds of pavements. The method is especially well-suited for construction of reinforced pavements.

The disadvantages of the method are many working phases as well as a high price. This method is still used in most countries which build concrete pavements although the slip-form method has gained more and more ground since the beginning of the 1970s.

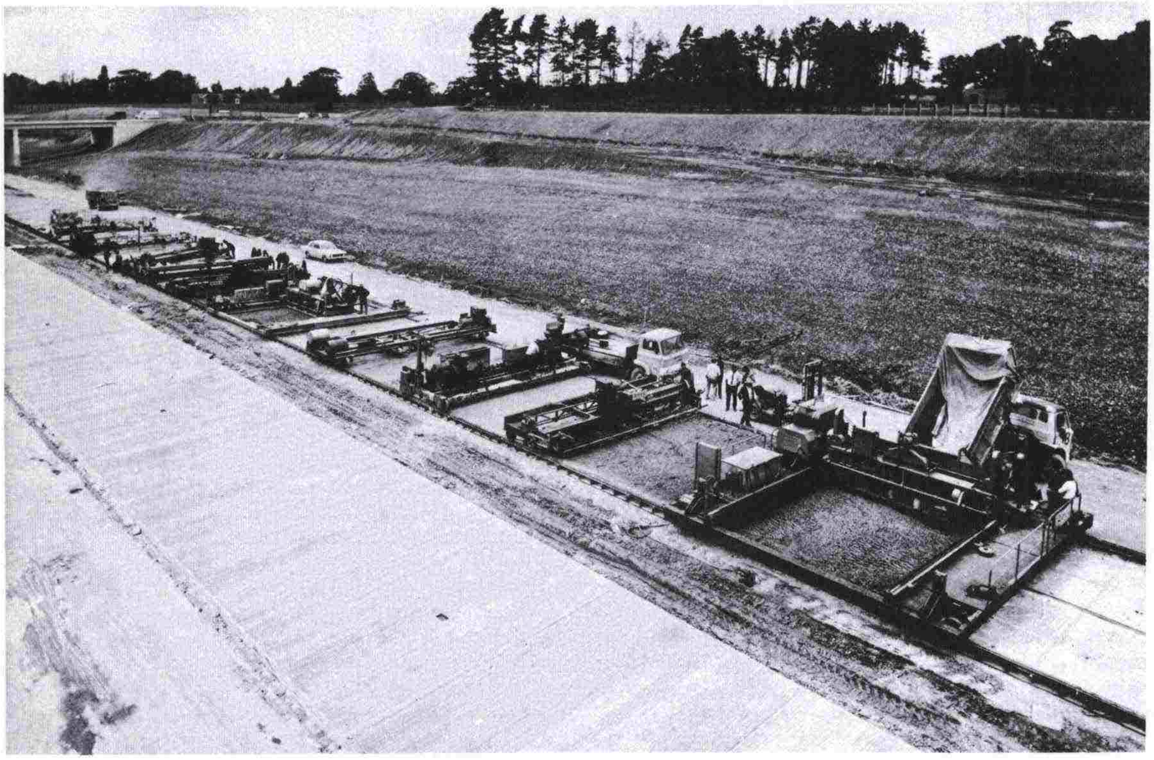


FIGURE B1-8. Concrete pavement under construction using the fixed-form method /14/

B 1522 The slip-form method

In the slip-form method a pavement is laid with a multi-function machine (Figure B1-9) which moves on own wheels or catepillar wheels. No side forms are used. Separate equipment is needed only for curing and texturing the surface and forming the joints. The method is most suitable for construction of unreinforced pavements; installation of dowel bars can also be made by means of a paver. A good capacity, flexibility in different working conditions and economy are regarded as advantages of the method. The slip-form method is a modern and popular method in concrete construction. A greater risk for uneven surfaces can be regarded as a disadvantage. Special attention must be paid to the construction work.

The working phases essentially affecting the final quality are studied in the following.

B 153 The most important working phases

B 1531 Production and hauling of the mix, Figure B1-10

The material should be mixed as near the site as possible in order to keep the hauling distances of the mix short and to avoid drying and segregation of the mix. The capacity of a mixing plant must be so great that a paver has a continuous availability of material. In a two-lane pavement construction this generally means a mixing capacity of 120...150 m^3/h.

Due to the requirement for equal quality only forced-action batch mixers are acceptable. When the mix is transported in open trucks the load must be protected from drying.

The quality of the mix must be followed with tests made during the spreading phase and by the eye. Unsuitable concrete must be removed before spreading if possible.

B 1532 Spreading of the mix, Figure B1-11

A paver should be operated at a slow (1-2 m/min) constant speed without stops. The screw should in front of the machine must be full of mix all the time. An even spread of the mix at the total width of the machine is ensured except by the screw auger itself also by a smoother moving in front of the auger (in the foreground in the middle in Figure B1-11). Steering mechanisms and sensor circuits at both sides of the equipment control the level of the beam and the direction of the equipment, Figure B1-12. If a laser beam acts as a sensor no steering wires are needed.

Spreading of the mix can also be made in two layers with two consecutive machines. To counterbalance the higher spreading costs the following advantages are achieved:

- the base mix can more cheaply be spread e.g. with an asphalt paver

- a firmer aggregate can be chosen for the surface layer

- the needed dowel bars and reinforcement can be installed on top of the base mix

- the requirements for evenness can more easily be met with thinner surface mix

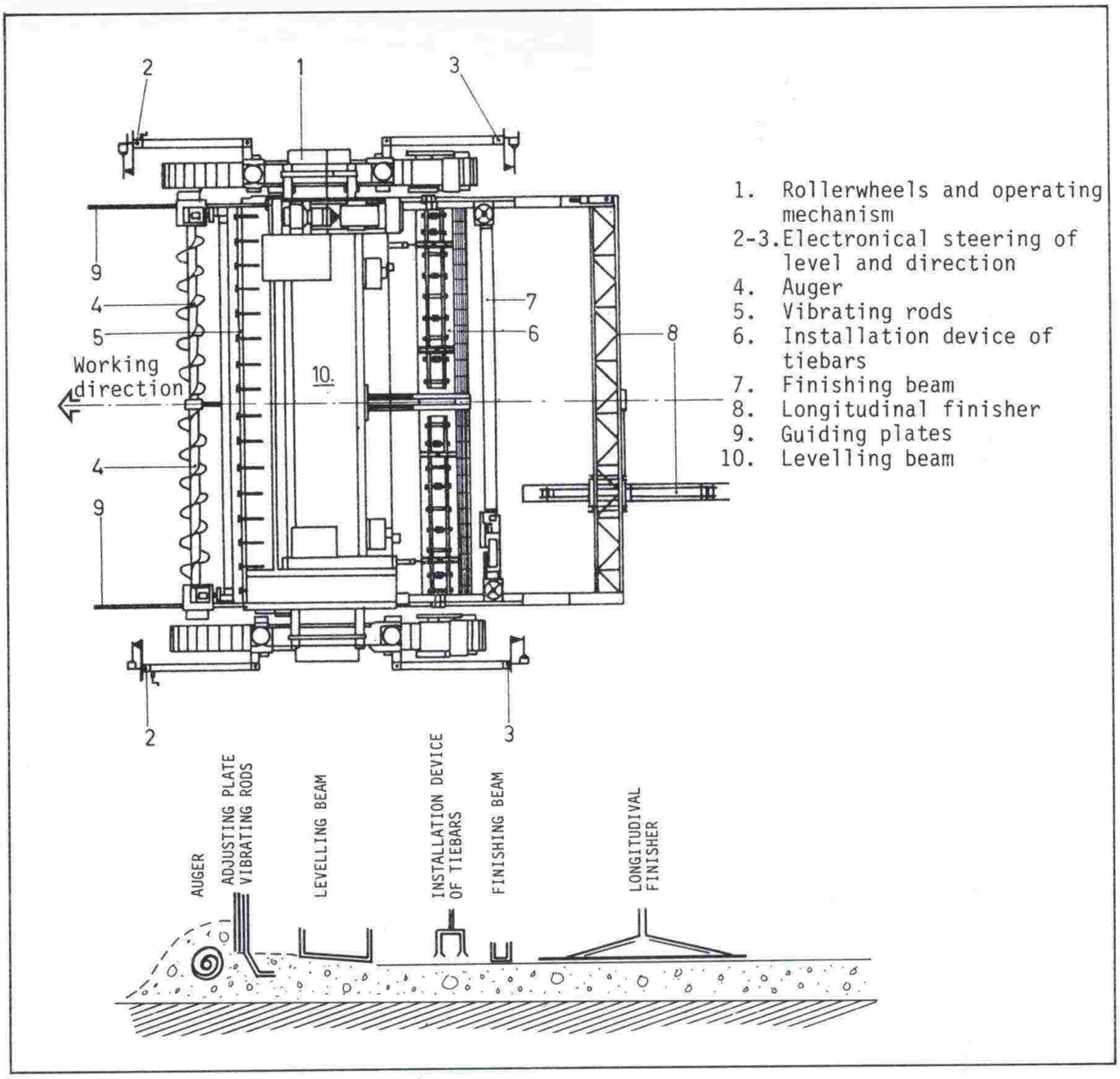


FIGURE B1-9. Structure of a slip-form paver

FIGURE B1-10. Working phases: production and hauling of the mix

FIGURE B1-11. Working phases: laying the mix (2-layer pavement)

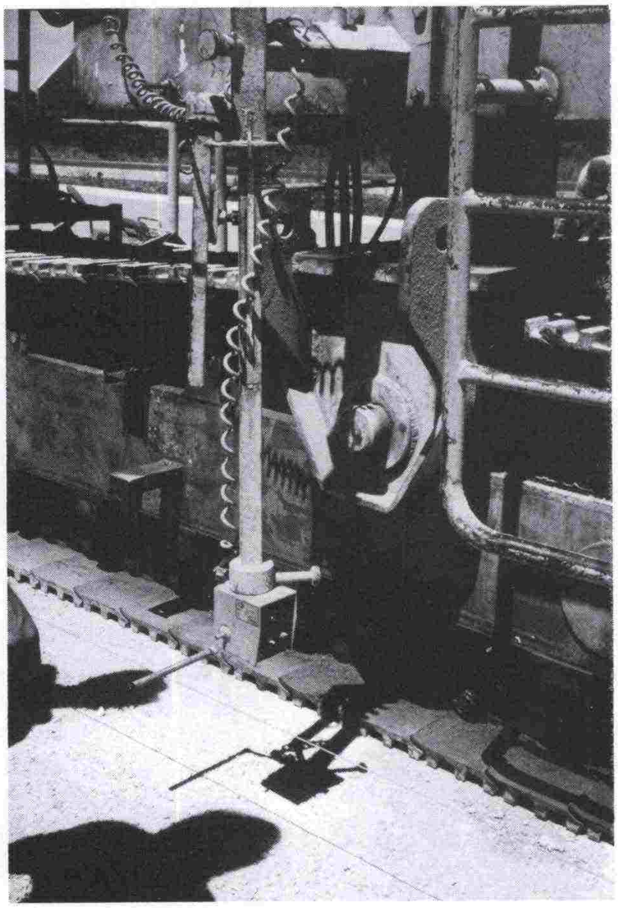


FIGURE B1-12. Working phases: level adjustment with steering wires

B 1533 Placing of dowel bars

The dowel bars of an unreinforced pavement can be placed beforehand by means of supports. The supports must be firmly attached to the ground in order to prevent the direction or position of the dowel bars from changing in connection with the casting phase. Because of the moving risk a new method has been developed. There a device attached to a slip-form paver automatically places (Figure B1-13) the dowel bars to the fresh mix. A finishing beam in a paver compacts and smoothens the surface also at the dowel bars. The automatic placing of dowel bars is very common although it is forbidden for the present for example in England, because there are still some doubts as to the placing precision as well as to the compactness and frost resistance of concrete just upon the dowels.

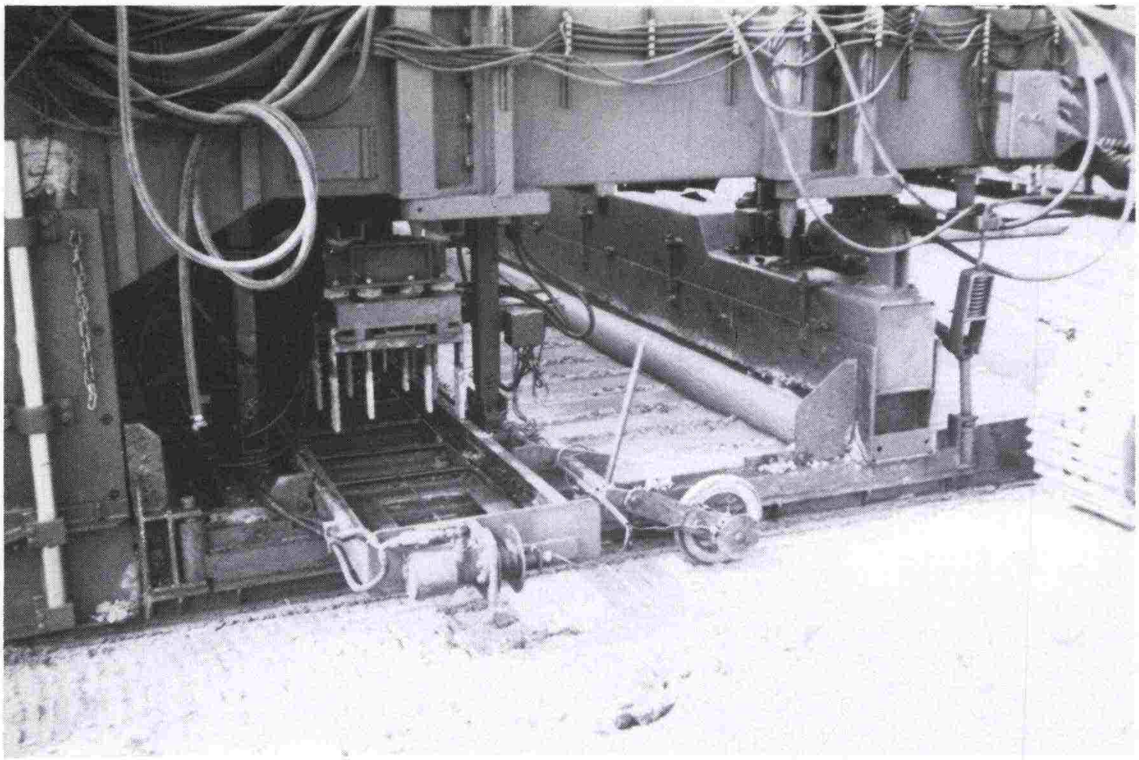


FIGURE B1-13. Working phases: placing of dowel bars with automatic placing device

B 1534 Finishing

Due to the vibration caused by a paver there can be some minor unevenness on the surface which weakens the driving comfort. This unevenness is removed by means of a longitudinal finisher of 3-4 m (Figure B1-14) which finishes the evenness with longitudinal movements. Also skew finishing beams overlapping the slab can be used.

B 1535 Texturing

Immediately after the paving procedure the surface is textured to improve the friction qualities. The most general working method is transverse brushing with a steel bursh (Figure B1-15). There are also other texturing methods as grooving (pressing transverse grooves with a roller wheel), longitudinal steel brushing and surface dressings. Exposion of the aggregate surface by removing the cement grout on the pavement surface could be a suitable method in the Finnish conditions. This is undertaken by spreading sugar liquor or other retarder preventing the cement from hardening on the surface. On the following day the cement grout can be removed by brushing and then the normal curing can be undertaken. The method is used in many countries; due to the studded tyre traffic it is especially suitable in Finland.

B 1536 Curing of the surface

Evaporation of water from the pavement mix has to be prevented. General methods are application of water or of plastic films. Special curing agents which are sprayed on the surface immediately after the texturing process (Figure B1-16) are widely used in road pavements.

FIGURE B1-14. Finishing the evenness with a longitudinal finisher

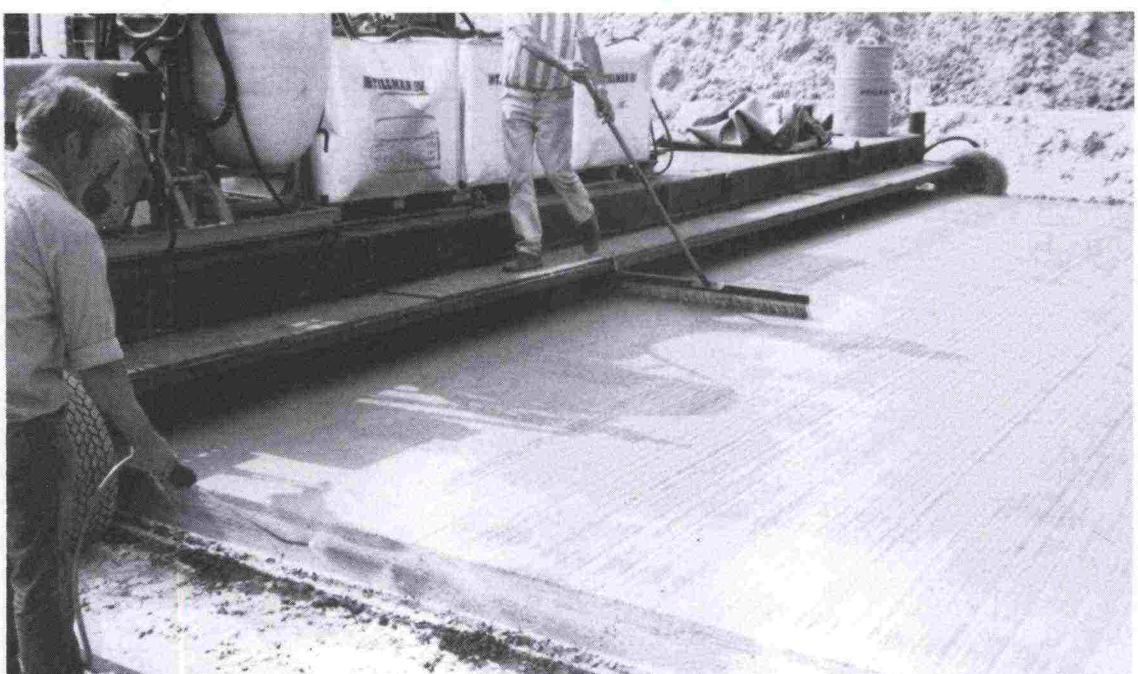


FIGURE B1-15. Working phases: wire-brush texturing

FIGURE B1-16. Working phases: spraying curing agents

B 1537 Sawing and filling of joints

In connection with the casting slab edges are marked to show the places of dowel bars. Transverse joints are sawn exactly at the middle of the dowel bars; this must happen as soon as the hardening pavement endures the sawing without breaking. The correct sawing moment is 5-15 hours after the casting depending among others on the weather, mix quality and pavement thickness. The sawing equipment must be sufficiently efficient (Figure B1-17) so that sawing can keep pace with paving. If sawing is undertaken too late, the shrinkage cracking of pavements occurs wildly somewhere else than at the joints and then the joints won't function in the planned manner.

A joint groove can be resized later by another sawing (see Figure B1-1), a joint strip is installed in a joint spacing (Figure B1-18) and the joint is filled with hot bitumen (Figure B1-19). When using silicone or other two-component joint sealants a better water resistance of joints can be expected but the price is considerably higher than that of hot bitumen mixes.

B 154 The most common working failures and their consequences

Figure B1-20 shows a summary of the most common working failures and their consequences in the construction of concrete pavements. These working failures have such a fatal effect on the evenness and durability of a pavement that everything possible has to be done to prevent them. A skilled working group familiar with the work and equipment and an up-to-date labour management are in the key position.

FIGURE B1-17. Working phases: sawing transverse joints

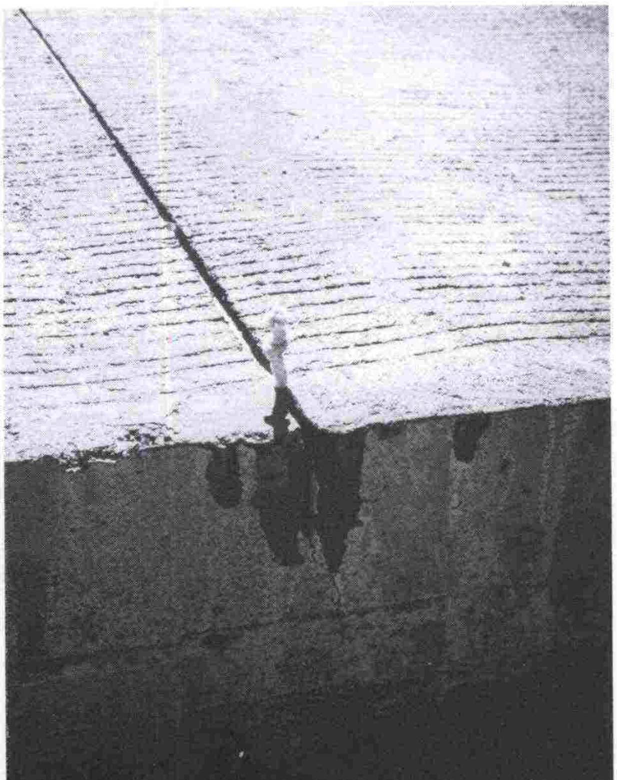


FIGURE B1-18. Working phases: installation of a sealing strip

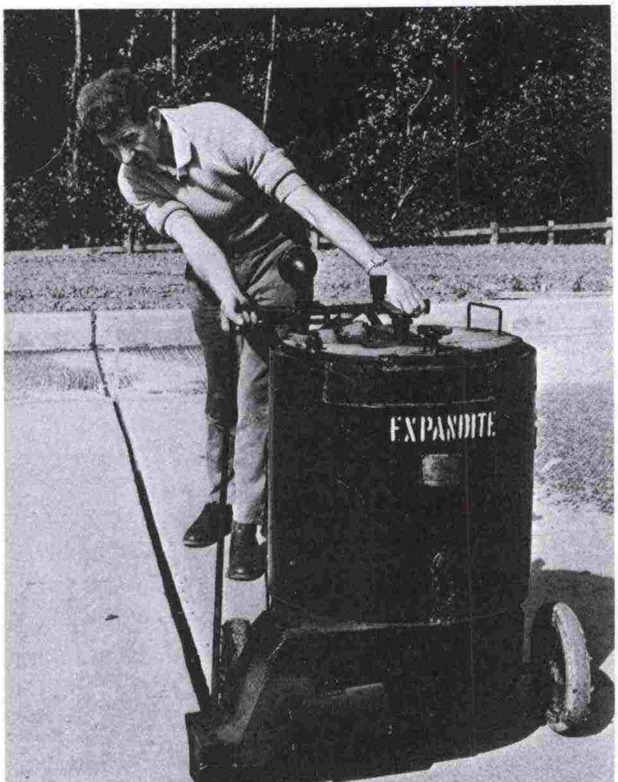


FIGURE B1-19. Working phases: sealing a joint groove with hot-poured sealant

Working failure	Consequences on the pavement
- the mix too dry	open spots, need for patching
- the mix too damp	edges break down
- too small mixing capacity	paver keeps stopping, unhomogeneousness, unevenness
- uneven or soft base	unevenness
- an inaccurate installation of dowel bars	cracking, spalling at joints
- lack of longitudinal finisher	'hitting' unevenness on the entire pavement
- inadequate brushing	slippery when wet as new
- too little curing agent	chazing on the surface, the strength and salt-scaling frost resistance remain low
- a delayed sawing	wild cracking, joints do not function
- dust or dirt in joint grooves	joint sealant does not stick to the joint walls, the joint is not waterproof

FIGURE B1-20. The most common working failures and their consequences in the construction of concrete pavements

B 16 COSTS AND COMPETITIVENESS OF CONCRETE PAVEMENTS

B 161 Construction costs

Due to the fact that only a few concrete pavements have been built in Finland no current market price can be given. A rough price estimation and a cost structure of unreinforced concrete pavements made on the basis of several publications can be seen in Figure B1-21 /21, 23/. The cost structure of asphalt pavements is similar. The share of binders is about one third of the total costs for both types of pavements. The price per ton of concrete and asphalt pavement - and thus the price per m^2 for equally thick pavements - is almost the same in Finland; abroad the price per ton of asphalt is calculated to be somewhat higher with the present price relations of binders.

The pavement price per m^2 is naturally dependant on the thickness, also many project and local aspects affect the price. A comparision of prices per m^2 of concrete and asphalt pavements is justified only when costs of corresponding structures are compared. In Finland and in many other countries asphalt structures have been built in many phases and thus concrete structures appear to be clearly more expensive due to greater initial investments. When comparing same material amounts or corresponding structures (the final situation) cost differences are small. An example of this is shown in Figure B1-22 with the prices quoted in 1985 in Colorado USA /17/, where the competition between concrete and asphalt pavements seems to be purest. The pavements concerned are unreinforced slabs without dowel bars and with skew joints.

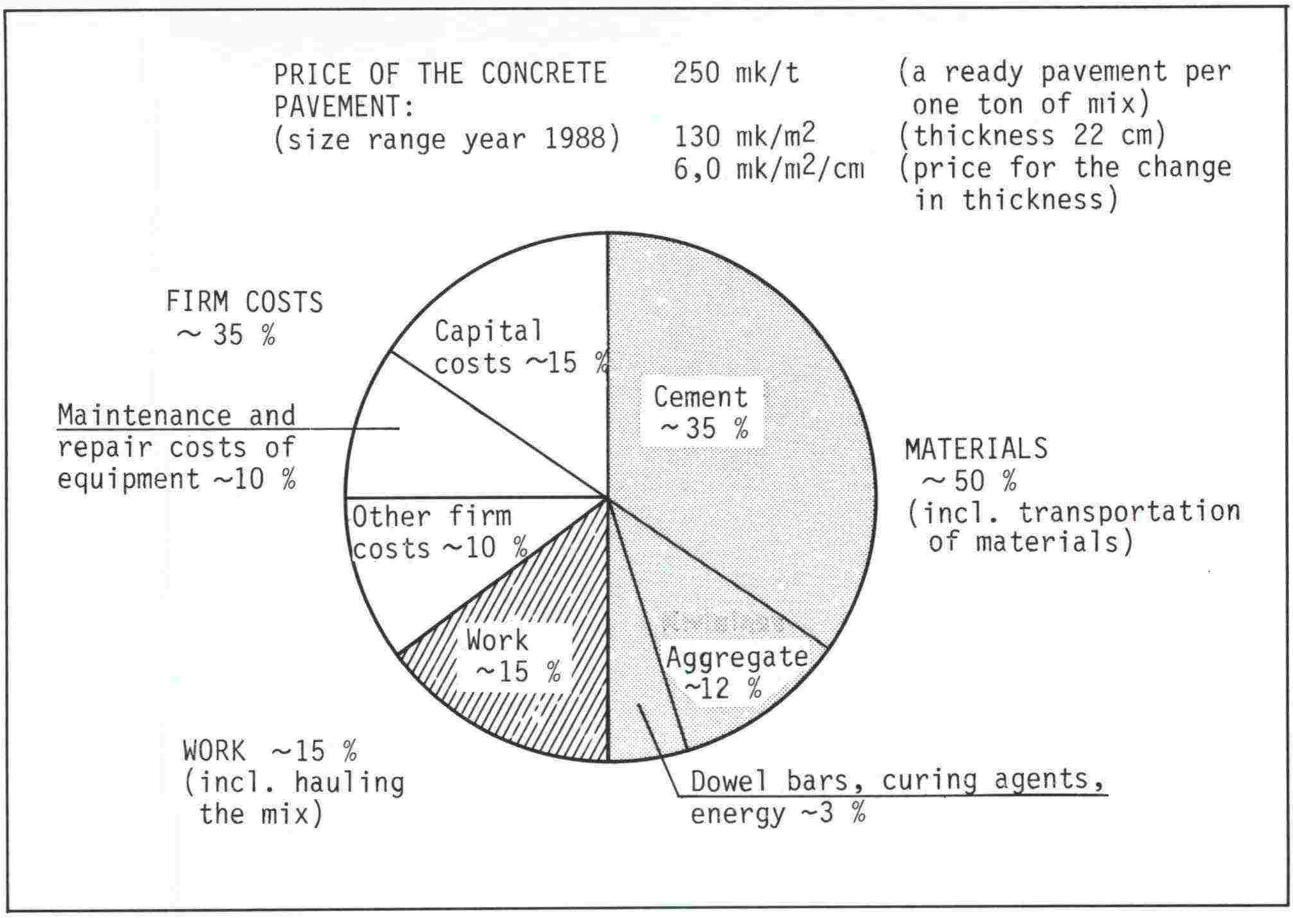


FIGURE B1-21. Price and cost composition of concrete pavements in Finland /22,23,29/

B 162 Life-cycle costs

The high initial costs of concrete pavements will be compensated by the following advantages: a longer and more reliable service life and smaller maintenance costs. This general statement can be proved as figures by comparing long-term costs of structure and pavement alternatives. The comparison is made by estimating, except construction costs, also maintenance and repair costs of the alternatives in the course of a certain observation time. All the costs incurred during the service life of the pavement will be discounted at their present value. Thus also an average annual cost for each alternative under observation can be defined; together with the total costs this cost is suitable for a characteristic when comparing the alternatives. A principle for a comparision of long-term costs has been presented in Figure B1-23. Figures B1-24 and B1-25 give examples of the applicability of the investment calculaton. The method is recognized and more and more increasingly used in the study of pavement strategies. Its reliability is dependent on how good experience and knowledge is available of the service life and costs of different rehabilitation measures. Furthermore, the comparision is influenced also by many factors at discretion such as the length of the observation period, the amount of the terminal value in different alternatives and the interest rate used. Due to these uncertainty factors investment calculations - as all forecasts - must be used as good instruments for decision-making without letting them have too definite a role in the final decision.

STRUCTURAL EQUIVALENCE:

$SN = \sum c_i \times d_i$

SN = structural number

d_i = layer thicknesses in inches

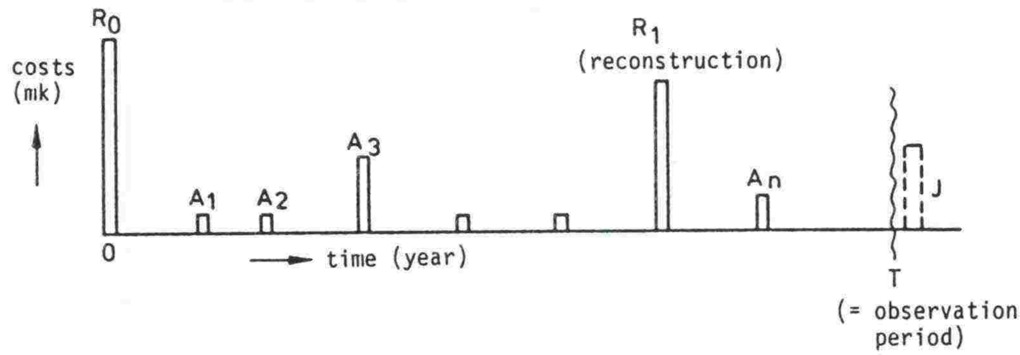
c_i = structural equivalence of the layer

c = 0,40 for asphalt
= 0,30 for bitumen gravel
= 0,12 for crushed gravel
= 0,50 for concrete

STRUCTURES AND COMPARISON PRICES:

PROJECT No.	Structural alternatives offered	Structural number	Offered price US$/sy (mk/m²)	Decision
A. (5600 m²)	a) 7½" concrete b) 8" asphalt c) 5" asphalt + 10" crushed gravel	3,75 3,20 3,20	13,17 (68,52) 12,68 (66,00)	Concrete was chosen; total costs least expensive
B. (7600 m²)	a) 7" concrete b) 8½" asphalt c) 4" asphalt + 12" crushed gravel	3,50 3,40 3,04	17,00 (88,45) 13,95 (72,60) 11,25 (58,55)	Asphalt (c) was chosen; concrete would have become 18 % more expensive
C. (36400 m²)	a) 7" concrete b) 8" asphalt c) 4" asphalt + 11½" crushed gravel	3,50 3,20 2,98	12,55 (65,30) 13,20 (68,70) 12,75 (66,35)	Concrete was chosen; is 4 % less expensive than the following
D. (11200 m²)	a) 6½" concrete b) 8" asphalt c) 4" asphalt + 8" crushed gravel	3,25 3,20 2,56	15,83 (82,40) 14,72 (76,60) 12,16 (63,30)	Concrete was chosen; although it was 12 % more expensive

FIGURE B1-22. Comparision principles and prices in Colorado in 1985 /17/



- construction costs R_0 are estimated
- maintenance, rehabilitation and reconstruction times are estimated as well as the costs of these measures $A_1 - A_n$, R_1
- the terminal value (J) of the studied structure is estimated at the observation period (T)
- the present values of all the measures are calculated using the formula $A' = a \times A$, $a = (1 + p/100)^{-t}$

  p = interest %
  t = the year the measures were completed

- the present value of the costs is calculated

$$K = R_0 + \sum_{1}^{n} A'n + R'1 - J'$$

- THE AVERAGE ANNUAL COST is calculated

$\underline{V = b \times K}$ b = annuity coefficient $= \dfrac{p/100}{1 - (1 + p/100)^{-t}}$

- the calculation is performed with several maintenance strategies of the pavement structure and the least expensive alternative regarding annual costs is selected

<u>NOTES:</u>
- the observation period T should be so selected that the reconstruction of all alternatives can be included; when the concrete pavement is included in the comparision, T is usually selected between 20...40 years
- the terminal value is the value of the pavement on the basis of the remaining service life and its value in the reconstruction (as crushed aggregates, as a base)
- traffic inconveniences caused by the measures can be included by adding vehicle and time costs to the cost estimate. This is generally done only with great traffic flows (ADT > 40000)
- the level generally used in public investments should be chosen as an interest level. Also other interest percents are used.
  A higher interest level favours construction by phases.
- inflation is ignored
- the values for coefficients a and b can be found in tables

FIGURE B1-23. Performance of investment calculations

Tabell 1 NUVÄRDE AV FRAMTIDA UNDERHÅLLSKOSTNADER

Kalkylperiod 30 år
Kalkylränta 5 %
Inflation 0 %

Åtgärd	År	Kostnad kr/m²	Nuvärdes-koefficient	Nuvärde
Asfalt HABD				
Justering 10 kg/m²	5	5	0.7835	3.92
Justering 10 kg/m² + 60 HAB12T + 90 HABD	8	55	0.6768	37.22
Justering 10 kg/m²	14	5	0.5051	2.53
Justering 60 kg/m² + 90 HABD	16	50	0.4581	22.91
Justering 20 kg/m²	22	10	0.3418	3.42
Justering 60 kg/m² + 90 HABD	25	50	0.2953	14.77
			Summa nuvärde kr	84.77
			Årskostnad kr/m²	5.40
Asfalt HAB				
Justering 10 kg/m²	6	5	0.7462	3.73
Justering 20 kg/m² + 90 HAB	8	39	0.6768	26.40
Justering 10 kg/m²	14	5	0.5051	2.78
Justering 30 kg/m² + 90 HAB16T	16	44	0.4581	20.16
Justering 20 kg/m²	22	10	0.3418	3.42
Justering 30 kg/m² + 90 HAB16T	25	44	0.2953	12.99
			Summa nuvärde kr	69.48
			Årskostnad kr/m²	4.43
Betong				
Fogåtgärder	7	1	0.7107	0.71
Plattjustering + fogåtgärder	14	1	0.5051	0.51
Planfräsning + fogåtgärder	20	32	0.3769	12.06
			Summa nuvärde kr	13.28
			Årskostnad kr/m²	0.85

Underhållskostnad enligt 5 årsplan drift 2.4 kr/m² och år

Tabell 2. ANLÄGGNINGS- OCH UNDERHÅLLSKOSTNADER FÖR ALTERNATIVA ÖVERBYGGNADSTYPER TILL E18 ENKÖPING - BÅLSTA

Lager	KR/m²	Lager ingående i resp. överbyggnadstyp								
		1 a	1 b	2 a	2 b	3	4	5	6	7
HABD 70 kg	24									x
" 90 kg	30	x		x				x	x	
HAB 60 kg	20	x	x							
" 80 kg	26							x	x	
" 90 kg	29									
" 100 kg	32		x		x					
Betong 20 cm	100					x	x			
AG 110 kg	31	x	x	x	x					x
Bitumenbunden bergkross 10 cm	15	x	x	x						
" 11 cm	16				x					x
Slurry 0,5 cm	6			x						x
CG 15 cm	40					x			x.	
CM 15 cm	50						x	x		
fiberduk	5						x	x		
Justering 10 kg/m²	5	x	x							
Anläggningskostnad kr/m²		101	103	82	79	140	155	111	96	77
Nuvärde framtida underhållskostn kr/m²		85	70	85	70	13	13	85	85	85
Totalkostnad kr/m²		186	173	167	149	153	168	196	181	162
Anläggningskostnad för objektet Mkr		37	38	30	29	48*	52*	41	35	28
Underhållskostnad för objektet Mkr		31	26	31	26	12	12	31	31	31
Total kostnad för objektet Mkr		68	64	61	55	60	64	72	66	59

*)Inklusive kostnad för ökad schakt 1.6 Mkr och för extra grundförstärkningsåtgärder 2.4 Mkr

Trafikklass 5 Medelköldmängd 300-400 d°C

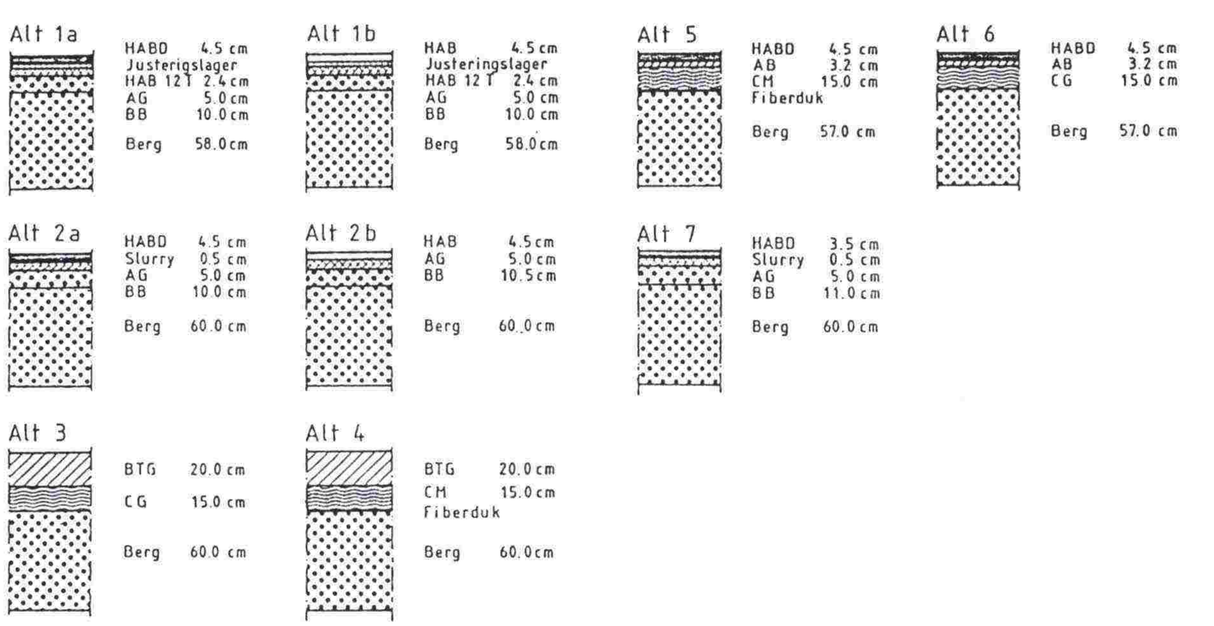


FIGURE B1-24. Cost comparision of Enköping-Bålsta structural alternatives (Vägverket, Sweden 1986) /27/

Main road 1, CEMETERY - LADJAKOSKI
INVESTMENT CALCULATION

Mr 1, Cemetery - Ladjakoski
Turku - Kaarina
(5400 m, 2 x 2 lanes AAT
10000...30000 vehicle)

Preliminary designs of a
white-topping concrete
pavement / 1987

CONCRETE PAVEMENT ALTERNATIVE | ASPHALT PAVEMENT ALTERNATIVE

v.	Measure	Cost	disc.int	present value	v.	Measure	Cost	disc.int	present value
			5 %					5 %	
1989) 0	Constr. of concret pavement (20 cm dowel bars)	13.500.000		13.500.000	0	Ab 20/120+levell. 16/25	3.160.000		3.160.000
1					1				
2					2	thin overlay	1.350.000	0,9070	1.224.000
3					3				
4	Joint repairs, (partly)	90.000	0,8227	74.000	4	thin overlay	1.350.000	0,8227	1.110.000
5					5				
6					6	Ab 20/120+levell. 16/25	3.160.000		
7					7	protective concrete	220.000	0,7462	2.634.000
						of bridges + attachment	150.000		
8	Milling + joint repairs	2.200.000	0,6768	1.488.000	8	thin overlay	1.350.000	0,6768	914.000
9					9				
10					10	thin overlay	1.350.000	0,6139	829.000
2000) 11					11				
12	Joint repairs, (partly)	90.000	0,5568	50.000	12	thin overlay	3.160.000	0,5568	1.759.000
13					13				
14					14	thin overlay	1.350.000	0,5051	682.000
15					15				
16	Grooving + casting (partly)	2.900.000	0,4570	1.325.000	16	thin overlay	1.350.000	0,4570	617.000
17	Joint repairs				17				
18					18	Ab 20/120+levell. 16/25	3.160.000	0,4190	1.387.000
19					19	attachment	150.000		
20	Grooving + casting (partly)	700.000	0,3769	264.000	20	thin overlay	1.350.000	0,3769	509.000
2010) 21	Joint repairs				21				
22					22	thin overlay	1.350.000	0,3490	471.000
23					23				
24					24				
	Terminal value 50 % of the original investment	6.750.000	0,3190	16.701.000 - 2.153.000 14.548.000		Terminal value 50 % of the value a of addit. asphalt	2.300.000	0,3190	15.296.000 - 734.000 14.562.000

Va = x 0,0741 = 1.078.000 | Va = x 0,0741 = 1.079.000=

COMPARISION COST:

= the present value of the interest of 5 %

CONCRETE PAVEMENT	=	14.548.000 mk
ASPHALT PAVEMENT	=	14.562.000 mk

FIGURE B1-25. A Finnish example about the performance of life-cycle cost comparisions /28/

B 163 Competitiveness of concrete pavement

Only in a few countries concrete and asphalt pavements actually compete on the basis of the construction costs according to Chapter B 161. In addition to the midwest states in USA also Germany and England decide the pavement type principally on the basis of contract offers. It is more common that a decision on the pavement type is made either by technical or economical comparisions (Figure B1-24) or by acquiring a political decision for design alternatives. Criteria of choice have been unitized in many countries. Traffic flow limits, for example, are such criteria; The most heavily trafficked roads are designed as concrete paved roads, with smaller traffic flows costs are compared per each project (Austria, Spain). Also a high functional class of a road or a high design speed can be technical grounds for choosing a concrete pavement. A longer service life (2-3-fold) of the concrete pavement is generally taken into account in economical calculations. Life-cycle cost calculations contribute to improving the competitiveness of concrete pavement. In Spain durability is emphasized by allowing a 1,2-times higher price for a concrete pavement in initial cost comparisions. In certain countries regulatory quotas for different pavement types have been given: e.g. in certain states in USA 20 % asphalt, 20 % concrete, the rest is decided in a price competition; in Switzerland a target of 50/50-ratio was pursued in the construction of the main road network (realized 25/75): a concrete pavement share of 20 % of the new motorways has been the target in England.. It is also very common that local or industrial politics play an important role in these elections. Also the construction tradition seems to guide the preference of different pavement types in many countries.

A strict price competition is typical all over the world where both pavement types are in use; development and marketing of both asphalt and concrete pavements are strong. As to concrete pavements the most efficient weapon in the competition is not so much the low price as the technical superiority, high quality level and the long service life.

After the oil crisis a thorough study of the technical and economical competitiveness of concrete pavement was made in 1980 in the Finnish publication 'Betonipäällysteen teknillisistä ja taloudellisista käyttöedellytyksistä Suomessa' (Survey on the Technical and Economical Prerequisites of Concrete Pavement) by Anssi Lampinen, VTT, Road and Traffic Laboratory. /22/

B 17 CONCRETE PAVEMENTS IN DIFFERENT COUNTRIES

B 171 Concrete pavements in Finland

Totally 360.000 m^2 of concrete pavements were built in Finland in 1926 - 1939./26/ They were mainly situated on the entry roads in Helsinki and Turku. After the War about 160.000 m^2 of concrete-paved roads have been built in Finland, Figure B1-26. 130.000 m^2 of airfield pavements was built before the War and approximately 170.000 m^2 after the War. Actual concrete-paved runways for airfields have not been built in Finland; concrete is mainly used on aprons and at runway ends.

Although only a few concrete pavements have been built after the War, the aim has been to follow the rapid progress in their design and construction. The latest pavements are very suitable for our conditions, they are modernly designed with a good structural durability, Figure B1-27. Owing to the lack of equipment and craftsmanship

THE MOST SIGNIFICANT CONCRETE PROJECTS IN FINLAND	Length (m)	Width (m)	Thickness (cm)	Joint space (m)	Strength of concrete	Constr. year
YLIKYLÄ-PARAINEN highway 180, Kaarina, Parainen	12600	2x3,50	18	15	K32	1958 1959
KEHÄ III highway 50, Vantaa	720	2x3,75	20	5	K30-50	1971
PALOJÄRVI-OLKKALA Main road 2, Vihti	1100	2x3,75	20	5	K40	1973
KALKKITIE Street, Parainen	1943	2x3,66	20 18	5 4,5	K40	1981 1982
RAJANIEMI-LAKALAIVA Mr II 45, Pirkkala	2050	2x4,25	20	5	K40	1983
VILLÄHDE-NASTOLA Mr 12, Nastola	2450	2x4,25	22	5	K60	1984
KEMPELE-KIVINIEMI Mr 4, Mr 8, Oulu, Kempele	4040	2x4,50	22	5	K55	1990

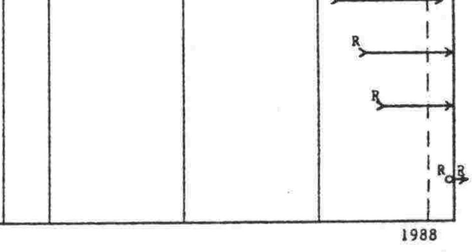


FIGURE B1-26. Concrete pavements in Finland (summary of the postwar pavements) /25/

The most significant concrete projects in Finland	Hw 180 Ylikylä-Parainen Kaarina, Parainen	Rr 50, radial r. III Veromiehenkylä -Tikkurila, Vantaa	Mr 2 Palojärvi-Olkkala Vihti	Kalkkitie, Parainen	Mr 45 Rajaniemi-Lakalaiva Pirkkala	Mr 12 Villähde-Nastola Nastola	Mr 4, Mr 8 Kempele-Kiviniemi Oulu, Kempele
ROAD - traffic - cross-section	1600-6400 (ADT 1980) 1,5 + 7,0 + 1,5	13000-25000(ADT 1979) 2,75 + 7,50 + 1,0	5200 (1983) 2,75 + 7,50 + 2,75	ab. 2000 (1982) 1,0 + 7,0 + 1,0	1900 (1984) 2,25 + 7,50 + 2,25	6700 (1984) 2,25 + 7,50 + 2,2	ab. 11000 (1990) 2,25 + 7,50 + 2,25
CONCRETE PAVEMENT - length - width - constr. year - repavement	12.600 2 x 3,50 1958, 1959 1984 with asphalt	720 2 x 3,75 1971 1985 with asphalt	1100 2 x 3,75 1973 1984 with asphalt	1206(-81), 737(-82) 2 x 3,66 1981, 1982	2050 2 x 4,25 1983	2450 2 x 4,25 1984	4040 2 x 4,50 1990
CONCRETE SLAB - width - thickness - reinforcement - joint space - joint struct. - dowelling - longitud. joint	3,50 18 cm ø 6 k/k 30...40 15 m, every third is expansion joint ø 3/4" 9 pcs L400 with tie bars, sawn, filled with bars	3,75 20 cm unreinforced 5 m 3 mm, no joint seal. no dowel bars ø 12 L750 k/k 100 bitumen coating in joint	3.75 20 cm unreinforced 5 m 4-5 mm, rubber.bit. no dowel bars,if stab. ø 12 L750 k/k 100 bitumen coating in joint	3,66 20(-81), 18(-82) unreinforced 5 m(-81), 4,5 m(-82) 3 mm/10 mm + sealant ø 25 L500, support ø 16 L900 k/k 1500 (81) ø 12 L800 k/k 1000 (82)	4,25 20 cm unreinforced 5 m 3 mm/10 mm+asb.strip ø 25 L500, support ø 12 L800 k/k 1000 sawn, overlaid	4,25 22 cm unreinforced 5 m 3 mm/10 mm+asb.strip + sealant ø 25 L500 support ø 16 L800 k/k 1000 sawn, overlaid	4,50 22 cm unreinforsed 5 m 3 mm/10 mm + strip + sealant ø25 L500 ø 16 L800 k/k 1000 will be sawn and filled
CONCRETE - binder - aggregate - additives - strength	Portl. (Rapid) max. 40 (64) K32	Portl. (Rapid.) max. 32 mm LPH K30, K40, K50	Portl. max. 32 mm Mischöl VR K40	Portl. max 32 mm Parmix, Melment L K40	Slag/Portl. 60/40 max. 32 (25) mm Parmix, Melment L K40	Salg/portl. 50/50 max. 32 mm Parmix H, Melment L K60 (91 days)	Slag/Portl. max. 32 mm K55 (91 days)
CONSTRUCTION - equipment - method - curing	ABC-paver, SGME-vibrator side forms brushing, rubbing, coating	ABC-paver, SGME-vibrator side forms brushing, rubbing, coating	vacuum concreting side forms brushing, rubbing, coating	CMI Super 200, slip-form nylon brush, Betokem	CMI Super 200, slip-form nylon brush, Curing, (laying with buckets)	CMI Super 200. slip-form nylon brush. Scandure 212, bag spreading	(foreign subcontractor) slip-form steel brush. trad. used curing agent
REMAINING STRUCTURE	normal 2-3 cm of levelling sand under the slab	subbase of macadam crushed stones or soil- cement	stab. subbase 420 cm thickness of the pavem. structure varying	crushed gravel, good load-bearing capacity	crushed rock, choke aggr. 0...100, 10-15 cm cr. grav. 0...65(35) 13 cm	sand 30...70 cm gravel 0-100, 25 cm cr.grav. 0...55, 7 cm	soil-cement 12 cm crushed stones 0-100 20 cm sand 86-136 cm
TESTS	20 pcs in 1958 (cem; aggr.; reinf.; brushing etc.	9 pcs (grading, strength, joints etc.)	10 pcs (thickn. and quality of unbound layers) also soil-cement	no tests (-81,-82 different structures)	7 pcs (KBet. etc.)	no tests	no tests
WORKING FAILURES	1958 test area unevener	- bad compaction - inacc. locat. of rails - no finishing beam - sawing delayed	- delayed sawing - lack of resularity follow-up dur. constr.	- concr. strength does not meet requirem. - many stops - sawing delayed - joint bask. in wrong positions	- subbase surf. uneven - joint bask. askew - too much joint sealant - stops. wait for mix - no finishing beam	- inaccurate sawing - baskets badly fixed - too many stops - equip. digged down to the base - too much handwork in finishing	
EXPERIENCES	- good wear resistance - too rough when worn-out - relatively many local consolidation repairs	- narrow joint bad - wear-res. 2,5-4 x hisher than in the adj. asph. - evenness and fricton worse - no damage at cement-tr. pavements	- long. joint must be sawn - 3,0 higher wear-res. against rutting - concr. noisier - 2-fold light reflect. - longit.cracks in the course of time	- spring/summer a good construction time - evenness not suffic. good - minor abrasion	- joint sealant loose - inadequate evenness - 2,08-fold wear-res. in in test track runs MANY WORKING FAILURES	- settlem. on shoulder - 3,1-fold wear-res. in test track runs - unevenn. hampers truck traffic	

FIGURE B1-27. Concrete pavements in Finland, specification /25/

all the projects except for the road Ylikylä - Parainen have failed as to evenness; (an unevenness figure of 230-290 cm/km measured by a bump integrator when the requirement would be <160 cm/km). In the construction of joints there are also severe defects.

The structure of post-war concrete pavements and experiences of them have been presented in detail in Figure B1-27. Also the structural data of the 4-km-long concrete pavement to be built near Oulu (Kempele-Kiviniemi) in 1990 are attached to the figure.

The temporary specifications of the Finnish Concrete Association (BY 18/1981) and the draft for design instructions from 1987 have been available for the design. The latest available data have been collected in the project specifications, last in the specifications of the Kempele - Kiviniemi road from 1987./29/

In addition to the above mentioned concrete pavements test pavements of rcc (totally about 40.000 m^2) primarily for the city street network have lately been built in Finland. The use of rcc mainly as a road and city street pavement will no doubt increase also in Finland. The problems of evenness and of other working techniques have so far prevented its wider use as a road pavement. However, rcc is already well known as a durable pavement of industrial yards and of other heavily loaded fields.

B 172 Concrete pavements in Scandinavia

1. Sweden

About 600.000 m^2 of concrete pavements have been built in Sweden before the War and about 1.300.000 m^2 after the War./27/ The two latest projects - E6 Vellinge - Malmö 1972, E4 Väla-Hyllinge 1978 - have been completed unreinforced and without dowel bars with slip-form pavers. Almost all the Swedish concrete pavements are situated in the most southern part of the country in Malmöhuslän. Except for the two above mentioned projects all concrete pavements have been paved with asphalt after their service level has decreased owing to slab damage and depression (see also Chapter B 283 and Figure B2-49). The rehabilitation problems of old pavements have caused certain reluctance to design new concrete pavements. On the other hand, development has closely been followed: concrete pavement is included in cost comparisons per project (e.g. Figure B1-24), test pavements are being built (e.g. Stockholm 1988) and design specifications (draft) have been published /30/.

2. Denmark

Most pre-war concrete pavements in Scandinavia have been built in Denmark, totally about 2 million m^2. The corresponding post-war figure is about 800.000 m^2. /20/ The post-war pavements are mainly on motorway sections around Copenhagen. The latest concrete pavement (7,1 km) is on a motorway in Falster from 1984. At the end of the 1960s almost 40 km of concrete motorway sections were built; in this connection big slip-form pavers and movable concrete plants were acquired. These pavements were, however, a disappointment-they were soon damaged and rebuilt with asphalt under the age of about 10 years. The damage was mainly D-cracking and polishing. The reason was confirmed to be the poor aggregate quality. In the 1970s only one test road of 4 km was built and this has served blamelessly. Also the pavement on the motorway built in 1984 functions irreproachably although some longitudinal cracking has occurred due to the compaction of the road structure.

The latest design specifications have been published in Denmark in 1983. /38/ Comparisions of concrete pavements are made only for heavily trafficked motorways in the prevailing price relations.

3. Norway

Sporadic concrete pavements were built in the surroundigs of Oslo in Norway before the War. After the War several concrete roads were built in Southern Norway as early as in the 1950s./21/ Especially in the county of Vestfold the road authorities used concrete as a pavement for the improved sections of the E18 Main Road. The latest E18 sections have been paved with concrete in 1979 (5 km) and in 1986 (6 km). About 20 km of concrete pavements have been built in Norway during the last twenty years. They are usually rather short and narrow 2-lane road sections. The use of concrete pavements also in tunnels and bridges is typical of Norway. Concrete has also been used on airfields and thus there are several modern slip-form pavers for different types of conrete pavements in the country.

Design specifications for concrete pavements are included in the official design standards and interest in further development has been constant, although there are only occasional projects. In connection with the latest pavement project (E18 Klinestad-Tassebekk in 1986) a significant research project to improve the wear resistance of concrete pavements was realized in cooperation with the different partners (see Chapter B 633). Great interest in the use of rcc as a tunnel and road pavement has been shown in Norway.

B 173 Concrete pavements in other parts of Europe

Concrete pavements have been used almost all over Europe since the 1920s. Amounts and proportions have varied in different countries but - except for Portugal, Greece, Italy and Turkey- concrete pavements have had and still have a strong foothold in the construction of high-quality motorway network all over Central and Southern Europe. Of the Eastern-European countries at least German Democratic Republic and Czechoslovakia have used concrete pavements and taken part in their development. Cooperation and exchange of experiences between different countries are very lively although the design practice differs according to the local conditions. A fresh example of the design cooperation is the preparation of the design standards of the TEM-project by order of UN. The TEM-project is a motorway project extending from Poland to Greece (Trans-European North-South Motorway Project) for which design specifications have been made out as an international co-operation. In connection with the project also the latest knowledge of the design and selection criteria of concrete pavements has been collected /35, 36/. In this context the European design practice won't be handled but the aim is to give a view of the general situation in those countries (Belgium, France, Spain, Germany, Austria, Switzerland, England) where information has been available.

1. Belgium

Belgium has the strongest and most variable tradition of concrete pavements in Europe. There are concrete pavements of different ages in all road and city street categories, totally over 21.000 km (21 % of the entire road and street network). The share of concrete pavements on motorways is 35 % (550 km), on other main roads about 15 % and on the municipal city street and road network 20 - 25 %. The latest types of pavement built on motorways are continuously reinforced pavements, unreinforced slabs are most commonly used on other roads. Older concrete pavements are of many different types and several different solutions must be used in their repair. Thanks to its reduced thickness a continuously reinforced pavement is less expensive in construction costs than a thicker unreinforced slab in a Belgian cost comparison. In a comparison of capitalized maintenance costs the costs of an unreinforced concrete slab are three times and those of asphalt six times the costs of continuously reinforced pavements. These unexceptional cost relations - and the great share of concrete pavements- are due to the strong position of the steel and cement industry in the country.

2. France

France was one of the first countries to adopt the slipform method and the unreinforced concrete pavements without dowel bars at the end of the 1960s. About 500 km of concrete paved motorways were built on the most heavily loaded main road network - the allowed axle weight being 13 t at the present. With time faulting of slabs and increasing cracking occurred. The experiences have led to structural experiments and development both in rehabilitation of old and in construction of new roads. On the basis of the French experiences the capitalized life-cycle costs of both pavement types are of the same size class. Today concrete is chosen due to the traffic flow or just to maintain competition. The share of concrete pavements of the entire road network is very small (about 1 %) which is due to the competition situation but also to a great extent to the fact that asphalt paved cement-treated structures are common and they are considered good and profitable structures-./22, 27/

3. Spain

The use of concrete pavements was very limited in Spain till the end of the 1960s. About 400 km of concrete-paved motorways (20 % of the motorways at the time) were built with slip-form pavers in the 1970s. Experiences of them have been good. Thus a new and extensive motorway programme (new motorways from Madrid to different directions to the coast) started in the mid-1980s will be mainly carried out as concrete pavements. Five big slip-form pavers have been acquired in 1986-87 and they work with the motorway programme at a speed of about 100 km a year. Totally about 2000 km of concrete pavements will be built in Spain by the beginning of the 1990s. A great part of them will be 4-lane motorways.

The use of rcc has long traditions in Spain. Thus rcc alternatives are included in the pavement structure standards, on inferior roads as such, concrete on the top, but on motorways paved with asphalt (an asphalt layer of 8 cm). Also some new motorways are built with this rcc structure. /27/.

4. Germany

About 63 million m^2 of concrete pavements were built in Germany before the Second World War. About 85 million m^2 of concrete pavements have been built in Germany after the War.

Concrete pavements have been concentrated on motorways; almost all motorways (about 2.500 km) built before 1960 were paved with concrete, the share has been about 30 % in later projects. The choice of a pavement is made as a contract competition in the paving phase and directly on the basis of prices. The material situation (availability of 'Rhein gravel') and local political issues effect on the fact that the use of concrete pavements varies much in different states (e.g. In Bayern 85%). Concrete pavements are constantly built by several contractors in Germany. Experiences of design and construction are good and the concrete roads are of high quality. Only slip-form technique is used. Pavements are equipped with dowel bars, they are unreinforced and short slabs are used./22, 27, 4)

5. Austria

A motorway network of 1.200 km has been built in Austria during the last 25 years. 85 % of these roads have been built as concrete pavements. The use of lightly-reinforced two-layer-pavements built with the fixed-form technique has been typical till the end of the 1970s. Later the use of asphalt spreaders in the construction of lower concrete layers and the use of slip-form pavers in the construction of even and compact surface layers have become general. Dowel bars are used in joints, but sawing is made narrow and no joint fill is used. The Austrian motorways are famous for their evenness and good condition. The studded tyre abrasion has been a problem in Austria (see Chapter B 282) but the problem is diminishing as a result of the fact that studded tyres are going out of use. /22, 27/

6. Switzerland

There are 350 km of concrete paved motorways in Switzerland or about 25 % of the main road network in the country. Totally about 5 million m^2 of concrete pavements have been built after the War. The main pavement type on main roads has been lightly-reinforced two-layer pavements, now the best type is considered to be the unreinforced one-layer solution. As in Austria also the Swiss concrete pavements are famous for their high quality. Corrosion of reinforcement bars and joint damage have aroused interest in maintenance questions now that the road network won't expand any more. /22, 27/

Swizerland has had a prominent position in developing the European model of modern concrete pavement and the working technique during the past decades.

7. England

England has one of the oldest concrete pavement traditions in Europe. Concrete pavements have been built mostly on motorways in connection with the post-war road construction. About 320 km of 4-lane concrete- paved motorways (unreinforced, short slabs equipped with dowel bars) have been built in the 1970s. This corresponded to a share of 15-25 % of the new pavements in different years /37/. Construction of concrete pavements has been active also in the 1980s and the pavement type is decided directly on the basis of contract offers. There are about 530 km of concrete pavements on main roads; this means about 5,5 % of the main road network.

The development work has been lively in England and the road standards have been kept up-to-date also as to concrete pavements /31/. The contribution in the international co-operation is significant.

B 174 Concrete pavements in the United States and Canada

Concrete pavements have been built nearly in all states since the turn of the century. In 1925 there were about 50.000 km of concrete pavements or 30 % of the total pavement length at that time. In 1985 there were 225.000 km of concrete pavements or 6,5 % of the total pavement length. About 50 % of the Interstate-highways - about 70.000 km built since 1975 - is paved with concrete, the share of concrete on the other main road network is about 15 %. The Interstate-network extends to all states and transfers about 21 % of the total traffic flow. Thus with the Interstate network concrete pavements became familiar also in those states where they were not built before. The use of concrete has been most common in California, Texas, Ohio and Pennsylvania. Concrete pavements are widely used also in such 'cold' states as in Minnesota, Wisconsin, Michigan and New York. /1, 2, 9, 10, 12/

After the great years of construction the pavement lengths won't increase any more and thus the attention is paid to development of maintenance projects and of repaving techniques (see Chapter B 281).

There are about 600 km of concrete pavements (about 2 % of the paved road network) in Quebec. 100 km of them have been built after 1970. The latest in 1984. The unevenness of the latest pavements and the decreasing service level of old pavements have caused a crisis of confidence between the public and the authorities. A committee, which has studied the matter, recommends in its report (1987) /8/ that construction of concrete pavements should be given up in Quebec for the time being, unless a certain minimum continuity can be secured to guarantee that the craftsmanship is retained.

There are about 1.000 km of concrete pavements in Ontario. They were purposefully built since the beginning of the 1950s during 20 years. After that construction of concrete pavements has been little mainly for three reasons: main road projects are decreasing, the price relation is unprofitable (concrete pavement 30 % more expensive) and a new mixed structure was developed: composite pavement with concrete base. This structure has been used in the most heavily trafficked motorway sections in the surroundigs of Toronto. An extensive test road section (15 km) was built in 1982 to study concrete slabs and pavement structure /33/. The results of this test will guide the design of the future projects. By means of test projects own specifications for different repair and rehabilitiation works have been developed in the 1980s.

There are about 900 km of concrete-paved roads in Manitoba-the western neighbour of Ontario - the climate conditions of which are more severe than in Finland. /11/ Main part of the roads are from the 1960s and 1970s, but new pavements have also been built annually in the 1980s. Also here the main attention has been paid to repaving and rehabilitation. The concrete network is estimated to remain in the present extension also in the future.

B 18 SUMMARY

In this section of the report bits and pieces of the history, qualifications, structures and extension and competitiveness of concrete pavement have been presented. The intention has been to give a general picture of concrete pavements emphasizing such views which can be considered of interest from the Finnish point of view.

The history of concrete pavements is long and its use has spread all over the world. Technically concrete pavements have been under constant development so that they can offer the best service level for both speedy and heavy traffic. Economically concrete pavements are often more expensive than other alternatives, but this concerns only construction costs; in the annual costs of their service life they are mostly an equally or even less expensive alternative. Concrete pavements really have all the technical and economical conditions to be investigated as an equal alternative when pavement decisions are being made.

CHAPTER B1 GENERAL SURVEY OF CONCRETE PAVEMENTS

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CHAPTER B2
REPAIR AND REHABILITATION OF CONCRETE PAVEMENTS

CHAPTER B 2

REPAIR AND REHABILITATION OF CONCRETE PAVEMENTS

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B 2
REPAIR AND REHABILITATION OF CONCRETE PAVEMENTS

B 20 INTRODUCTION

A good load-bearing capacity and evenness, a long service life and a small need for maintenance are often regarded as characteristics of concrete pavements. Concrete pavements of the first generations have been allowed to age in many countries without paying any attention to their maintenance. 'Build and forget it' was said to be a motto in the United States. And it is true that concrete pavements came up to the expectations - but only for a time. However, at the end of their design life or when approaching their equivalent axle load limit damage began to appear on pavements for different reasons: faults in joints, cracking, spalling, surface rutting, unevenness etc. The naked truth is that at the end also concrete pavements will be worn out by heavy traffic together with temperature and moisture variations. This has become increasingly obvious since the early years of the 1970s; there is a growing number of old and deteriorated concrete pavements in many countries and something has to be done to them. Reconstruction of all aging concrete pavements is economically an exceedingly heavy task and thus new means to extend the life of pavements are sought all over the world. Because the repair and rehabilitation techniques have not been under control, the life of concrete pavements has often been extended by patching and repaving with asphalt. The inconvenience of concrete pavement rehabilitation has resulted in reluctance to build new concrete pavements.

The 1980s has been the time for strong research and development activities to find new and more applicable repair and rehabilitation methods of concrete pavements and, also, to improve their long-term durability. The research work and the testing of methods are still going on in different parts of the world, but we already know that techniques can be found for repair and rehabilitation and that skilled design, construction and preventive rehabilitation are in a keyposition as to the long-term durability of pavements.

The fact that repair of concrete pavements is considered inconvenient is one of the reasons for their little use also in Finland. The traffic stress in Finland is smaller than generally in countries where concrete pavements are built but correspondingly the climate stress is greater in many ways. The studded tyre wear is also a significant threat to the durability of pavements in Finland. Thus it is very important that the uncertainty factors of the repair and rehabilitation of concrete pavements are investigated before wider application of cp in the Finnish conditions.

In this section of the report the emphasis lies on typical concrete pavement damage, their causes and repairing methods, on the applicability of rehabilitation methods in different circumstances and on the study of reconstruction of concrete pavements. This survey is based on foreign experiences and literature. Applicable solutions for the repair of a typical Finnish damage, rutting, are especially looked for.

B 21 CONCRETE PAVEMENT DEFECTS AND THEIR CAUSES

Stresses occurring in pavement slabs are caused by the following factors:

- traffic factors
  - a dynamic constant traffic load
  - unexceptional overloads
- traffic management
  - maintenance equipment and measures, e.g. salting
- environmental factors
  - temperature variations (frost, heat)
  - moisture variations (wet/dry)
  - frost-susceptibility of the subgrade
  - settlement of the subgrade

Although concrete pavements are designed according to the prevailing traffic and environmental conditions to endure all the above stresses, damage will gradually arise on the pavement. Also working and material failures will result in defects either immediately after the construction work or in the course of time when the pavement is aging. The damage types of concrete pavements can be divided into the following groups: 1) surface damage, 2) joint damage and 3) constructional damage.

The typical defects of jointed concrete pavements are presented in Figures 1, 2 and 3./1/ There are no joints in continuously reinforced pavements so that the description of the damage would also be somewhat different. Because continuously reinforced pavements are not built in Finland, only defects of jointed slabs will be dealt with here. All the failure types presented do not occur in the Finnish conditions because defects typical of different countries concentrate on certain groups depending on materials, traffic and environmental conditions. Also possible reasons for damage are described in Figures 1 - 3. Before taking repair actions the probable causes for damage should be found out. Only this way correct repair and rehabilitation methods can be selected and a long service time achieved for a pavement.

Type of defect	Description of defect	Possible causes	Damage caused
irregularity of the surface	irregularities on the surface, which can be proved by measurements or observed by the eye	- defective work performance - warping of slabs	- decrease in the service level
inadequate skidding resistance (not in Finland due to studded tyres)	road surface slippery	- polishing aggregate - inadequate or no surface texture - dirt, cement paste etc. in the surface	- danger for driving
inadequate drainage	road surface wet or it is ponding long after the rain	- drainage system choked up - insufficient gradient - ruts	- splashing of water - danger for aquaplaning
ruts	ruts at wheel paths due to abrasion of concrete	- studded tyre traffic	- danger for aquaplaning - danger for driving out of road
chazing, plastic shrinkage cracking	thin hair cracks on the slab surface	- inadequate curing - use of salt	- no effect on the service level - if not repaired leads to scaling
scaling	gradual weathering of the surface; loosening of cement grout and then of rough aggregates	- bad salt-scaling frost resistance - abundant use of salt	- reduction of the riding comfort - gathers water, which accelerates scaling when freezing
peeling	loosening of the thin repair layer of cement grout as large pieces	- too thin a layer - bad bondage - wrong mix proportion of the cement grout	- loosening pieces are dangerous - decrease in the regularity
pop-outs, pot holes	often round ravellings of different sizes otherwise on a healthy slab; are revealed soon after the construction	- unhomogeneous mix - bad frost resistance - reactions in the mix - corrosion of the dowel bars in the slab (if there are dowel bars)	- sharp holes - small ones do not require repair actions, greater ones must be at once repaired

FIGURE B2-1 a. Surface damage types of concrete pavements and causes for them

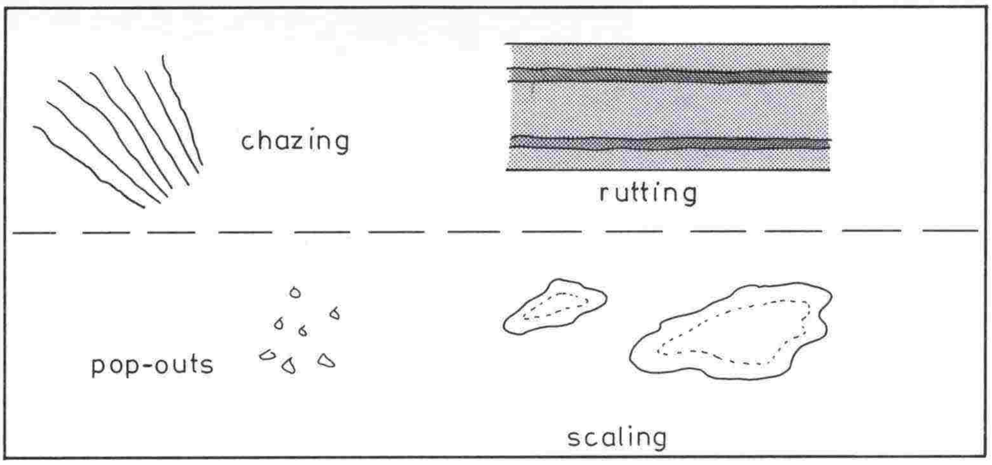


FIGURE B2-1 b. The most common surface defects of concrete pavements

FIGURE B2-1 c. Surface scaling caused by the action of frost and de-icing salts

Type of defect	Description of defect	Possible causes	Damage caused
leaking joint	lack of adhesion between the seal and the sides of the sealing groove; access of surface water to the joint	- bad joint sealant - old sealant - incorrect sealing groove dimensions	- the water in the joint will strain the slab in the course of time (note! freezing, pumping)
overflowing or scant joints	overfilled sealing groove or the groove is not sufficiently filled	- too much or too little joint sealant - bad quality of the sealant - incorrect sealing groove dimensions	- irregularity, joints are 'hitting' - untidiness - ingress of incompressible material into a scant joint which may break the joint edges
spalling	the slab spalls at the transverse or longitudinal joint; pieces of concrete will loosen from the upper edges of the slab	- weakness of the slab edges, fine-grained mix in the surface, too deep a transverse brushing at the joint - incorrect installation or locking of dowel bars - stones or incompressible material in the joint	- loosening pieces often result in an immediate need for repairs
cracking	the slab cracks in the vicinity of joints	- stones or incompressible material in the joint - incorrect installation of dowel bars	- danger for breaking of the slab - maintenace may be increased

FIGURE B2-2 a. Joint defects of concrete pavements and causes for them

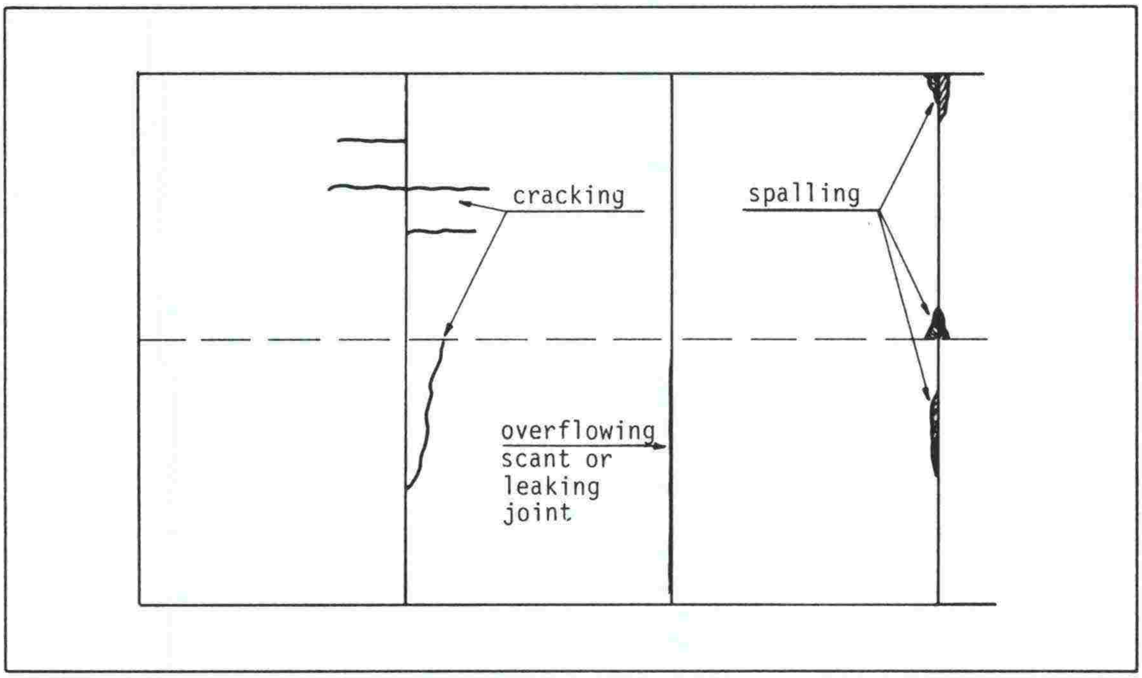


FIGURE B2-2 b. The most common joint defects of concrete pavements

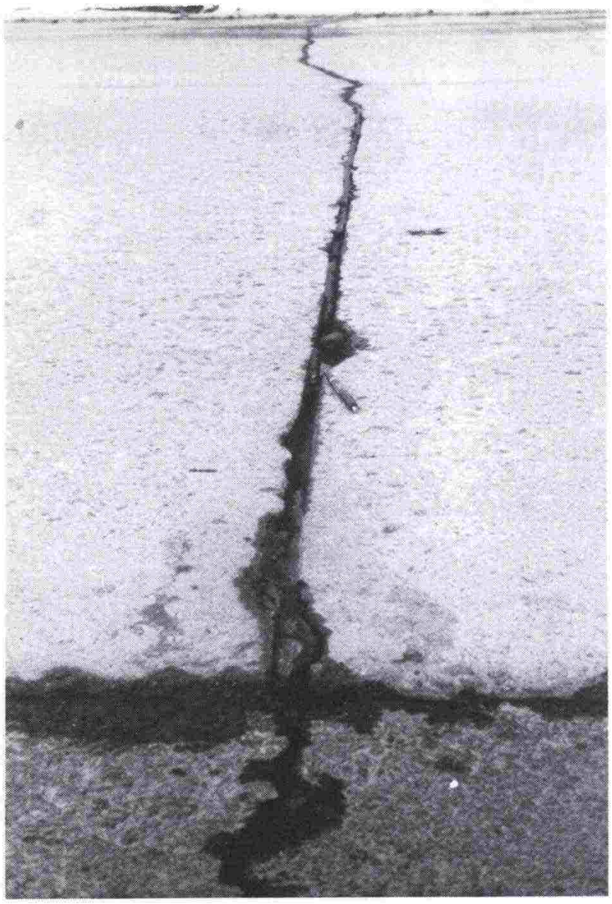


FIGURE B2-2 c. Mild cracking in a leaking joint

Type of defect	Description of defect	Possible causes	Damage caused
transverse cracking	a direct or skew crack extending from one slab edge to another	- short-dimensioned thickness - excessively long slabs - late sawing of joint grooves - fatigue of the slab - bad quality of concrete	- no direct damage to traffic - decrease in the load-transfering capacity in the course of time
longitudinal cracking	in direction of the medial joint, extends to several slabs	- excessively wide slabs - excessively low joints - incorrect location of the medial joint as to driving lines - settlement of the road edge - frost action of the road	- may cause detrimental differences in grading or the joint may open
corner cracks	a crack extending from the transverse joint to the medial joint or to the slab edge	- overload in the corner - defective joint seal - erosion in the base	- corner cracks deteriorate easily; danger for spalling
third stage cracking	the slab badly cracked diagonally, no adhesion between slab parts	- an overloaded, worn-out slab - inadequate load-bearing capacity of the base	- the entire pavement to be renewed immediately
faulting, stepping	the slab on the driving side settles and this causes bumps	- frost action - the base sensitive to erosion - lack of dowel bars	- driving comfort diminishes, "hitting" pavement - the slab will be damaged also otherwise
pumping	overflowing of water and soil material from transverse joints under a heavy vehicle	- the base sensitive to erosion - inadequate drainage - inadequate load-bearing capacity of the base	- cracking and stepping of the pavement
slab rocking	vertical slab movement under a heavy vehicle	- same causes as in the pumping	- leads to stepping
D-cracking	a series of advancing hair cracks in the vicinity of the joints and corners	- weathering caused by reactions of aggregates and cement and accelerated by water and freezing (not in Finnish aggretes)	- cancer of concrete which hopelessly destroys the slab
blow-up	destruction of transverse joints when the slab is raised at joints	- concentration of the compressive stress - incorrect structure of the expansion joint	- takes place suddenly and stops the traffic - requires replacement of at least two slabs
settlement, bump	detrimental longitudinal or transverse grading	- frost action or settlement of the base	- decreases driving comfort - the slab cracks

FIGURE B2-3 a. Structural defects of concrete pavements and causes for them

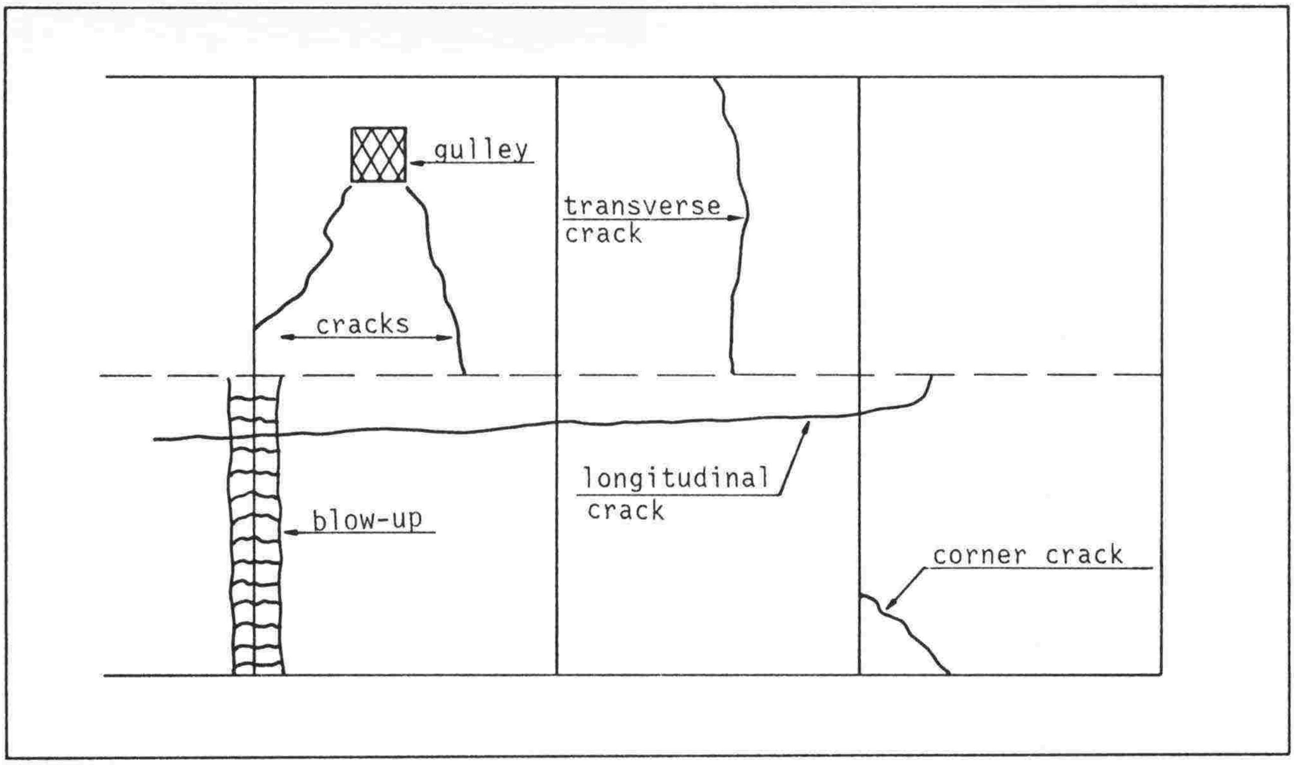


FIGURE B2-3 b. Most common structural defects of concrete pavements

FIGURE B2-3 c. A cracking concrete pavement

B 22 DAMAGE EVALUATION

Concrete pavement defects should be repeatedly evaluated for many important reasons:

1) to follow up the condition and damage process of the pavement,

2) to design instant or annual repairs,

3) to anticipate extensive rehabiliation projects and to design rehabilitaton actions,

4) to acquire material for development of design, repair and rehabilitation methods thanks to repeated evaluation of all concrete pavements.

Damage evaluation is based on observations by the eye and on measurements. The pavement is studied slab by slab and the results are written down on a form like the one shown in Figure B2-4. The evaluation is repeated annually or periodically. The damage type is stated, its extension is measured and the cause and severity are estimated. The extension of damage is measured with a measuring tape, deflections of a slab e.g. with a Benkelman-beam and the evenness with an evenness indicator. Precise instructions for damage evaluation have been pusblished in most countries /9, 12/, including e.g. instructions of measuring methods, severity definitions or even drawing marks.

B 23 REPAIR AND REHABILITATION METHODS

Repair of concrete pavement damage has generally been neglected everywhere, although there have been instructions for the most common failures e.g. for spalling, joint damage and scaling as long as concrete pavements have been built. Among others, the amount of handwork and the slowness of repair works have resulted in this neglection. Due to this neglection of repair works no development of craftsmanship and methods took place. Only in the 1970s - when demands for a better service level and the amount of pavements in need for rehabilitation increased- development of repairing methods to an up-to-date level started. Thus detailed instructions on repair of different kinds of defects have been published in most countries by the mid-1980s /1, 2, 3, 4, 8, 10, 11/. The methods have been tested in practice and also an increasing amount of experience of the durability of the repairs is now available. Because conditions vary in different countries, detailed specifications have to be drawn up in each country on the basis of their own field tests. The most common repairing methods extracted from certain foreign manuals are described as a basis for Finnish development and testing activities in this context.

B 231 Repair of joints

When joint sealant has loosened from concrete so that access of water into the joint is possible or when there is too much or too little joint sealant, the joint must be repaired (Figures B2 - 5). The repair is best carried out in spring or in autumn when the weather is warm and dry.

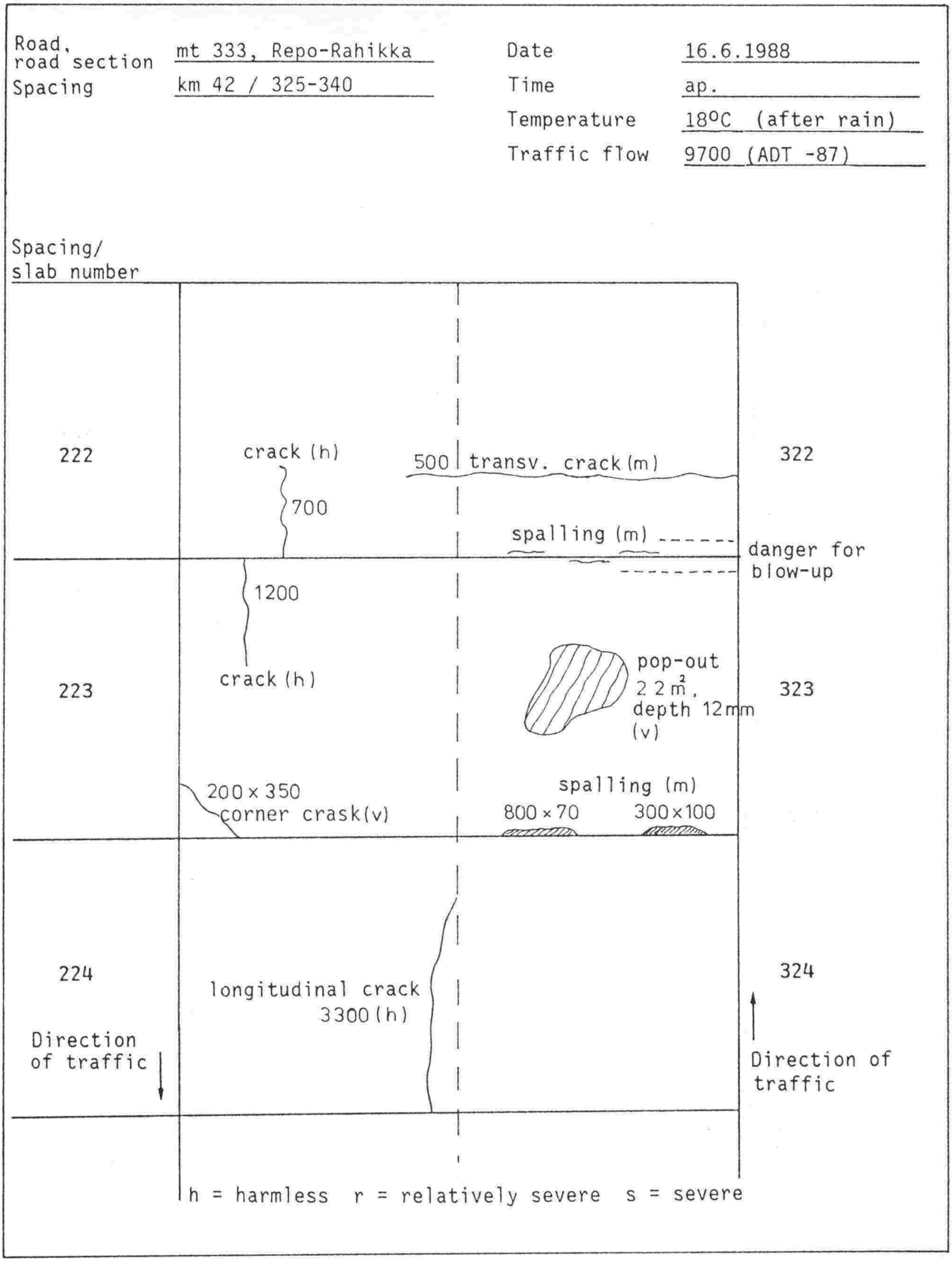


FIGURE B2-4. An example form to perform damage evaluation

The working stages are:

- removal of joint sealant and the sealing strip

- cleaning of the joint spacing by sand blasting or by resawing

- cleaning of the joint spacing with compressed air immediately before the filling

- installation of a new sealing strip

- treatment of the joint spacing edges with a solution that provides added cohesion

- pouring or placing of new joint sealant into the joint

Hot-poured sealants should be handled by means of a placing device supported to the pavement so that the joint is evenly and at one feed filled to a correct filling degree, Figure B2-6, and that no air bubbles are left in the joint.

Hot-poured rubberized bituminous joint sealants are generally used in Finland. Good experiences of cold-poured Silicon-mixes have been received in the United States. Two-component plastic sealants require great installation precision and standard conditions which make their use difficult in variable field conditions. The price of ready-made rubber profiles is high and the experiences of their use are varying so that they are not generally used in cold areas.

The repair of the joint must be extended to the total width of the slab. If the repair is done only by increasing sealant it does not secure the water tightness of the joint; thus the whole joint must always be repaired by means of the above mentioned working methods. Because joint sealants age in any case, all joints should periodically be resealed, a recommendable resealing cycle is 5 years.

B 232 Crack repairs

Hair cracks of less than 0,5 mm do not weaken the load-transfering capacity from one part of the slab to another, they do not drain surface water through the slab, either. These cracks are not repaired. Cracks of 0,5 - 1,5 mm will be sealed against access of surface water by means of a method described later. If the width of a crack is more than 1,5 mm, the load-transfer is lost and the slab or a portion of it must be renewed. These crack widths should be measured in winter.

Medium cracks (0,5 - 1,5 mm) will be sealed by making a groove at them, Figure B2-7, which will be filled in the same manner as a sawn joint. A device which can follow the winding crack must be applied in the grooving process, otherwise the repair won't succeed. After grooving the groove is cleaned, a round asbestos strip is installed in it and the groove is filled with proper joint sealant. The filling is often undertaken without a sealing strip. Sometimes light sealants are used in cracks so that the uniform outlooks of the slab would not be disturbed. Figure B2-8 presents a crack repaired in the above described manner where new damage, however, has taken place already.

Serious cracks (> 1,5 mm) are repaired according to Chapter B 234 by renewing the full-depth slab part. A single crack can also be repaired by installing dowel bars in it to restore the load-transfer, Figure B2-9. Steel locking device installed in holes drilled at the crack can also be used.

FIGURE B2-5. Overflowing expansion joint

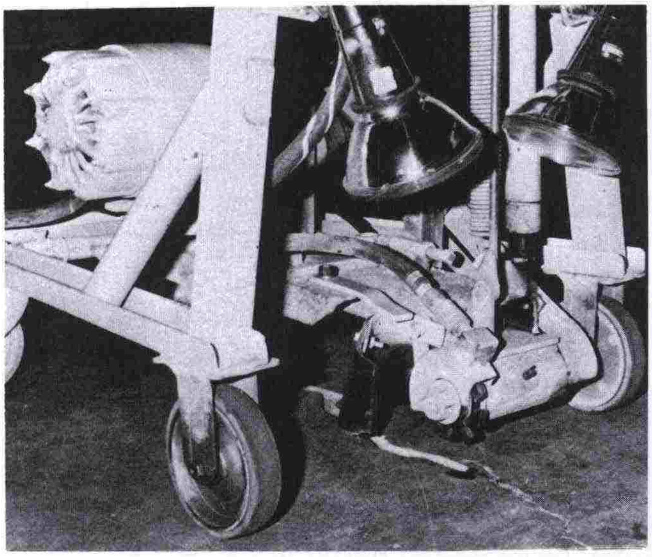


FIGURE B2-7. Repair of cracks by scabbling /21/

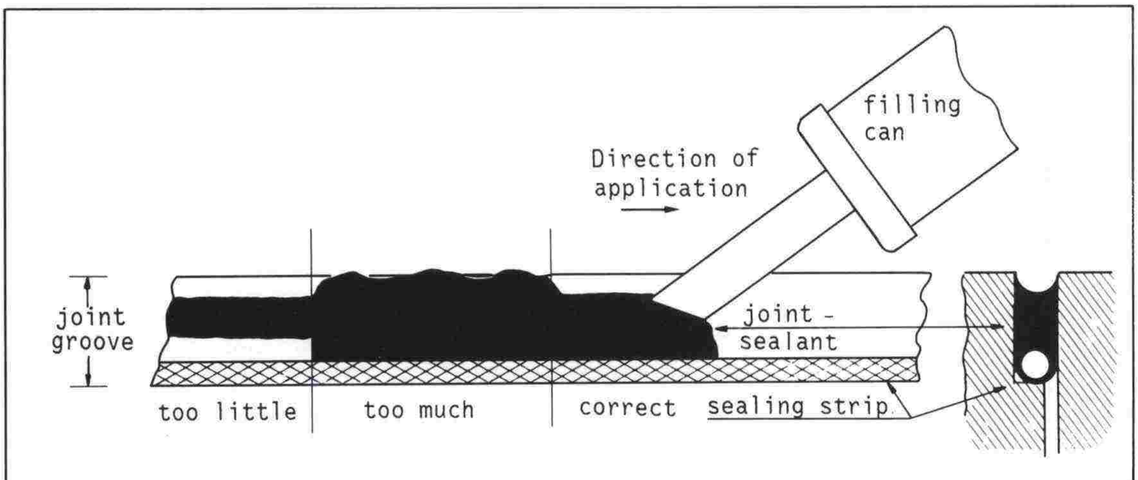


FIGURE B2-6. Filling a joint /1/

B 233 Patching

In this connection patching means repair of spalling or of other local surface damage by removing part of the pavement thickness at the damage and by repairing it with new concrete mix. Patching includes the following working phases:

- A rectangular patching area is marked out around the repair, at least 150 mm x 150 mm, and a minimum of 50 mm beyond all unsound concrete.

- Locating grooves are sawn at the patch edges

- The hardened concrete at the damage is grooved or milled and then cleaned out until all unsound concrete is removed, Figure B2-10. The minimum thickness of the patch is 10 mm. If the patch thickness exceeds 10 cm (over 40 % of the slab thickness) the repairing method must bechanged and a full-depth repair according to Chapter B 234 must be made.

- The area to be made is cleaned with compressed air.

- The area to be patched is carefully dampened if the patching is made with concrete mix. If the patching is undertaken with epoxy or latex mix, the surface treatment is carried out on a dry surface.

- Patching mix is spread to a 20 % surcharge then the surface is finished and protected from drying.

- If patching is made in the vicinity of a joint, a temporary filling in the joint is used which is removed after the hardening of the mix and the joint spacing is filled in a normal way.

An example of a succesful patching is shown in Figure B2-11.

Cement grout is used as patching mix in patchings of not more than 30 mm (1 portion cement, 3 portions sand, w/c = 0,45); concrete the maximum grain size of which is 10 mm (1 portion cement, 2 portions sand, 2 portions D 2,..10 mm, w/c = 0,45) is used in patchings of more than 30 mm. The durability of the patch can be improved by compensating part of the water with latex (about 9 kg latex/50 kg cement). A quickly hardening and durable patch can be achieved with two-component plastics like epoxy. Only small patches can be made with epoxy (10 portions sand, 1 portion epoxy), because, due to the disparity between the thermal characteristics of concrete and epoxy, good bond to the old concrete cannot be ensured if large patches are used. Inspite of the high price epoxy is generally used in repairing small joint cracks, because the protection time needed is short (3 hours). Cement-based patching mixes are less expensive, but they require a longer protection time. Asphalt is also well-suited for temporary patchings. Also with asphalt the damage area must be carefully cleaned in order to get a clean and durable patch.

B 234 Full-depth repairs (replacing a portion of slab or an entire slab)

It may be necessary to replace a portion of slab or the entire slab for example owing to severe cracks or to the destruction of the slab at a joint due to a blow-up. The main working phases of the full-depth repair are:

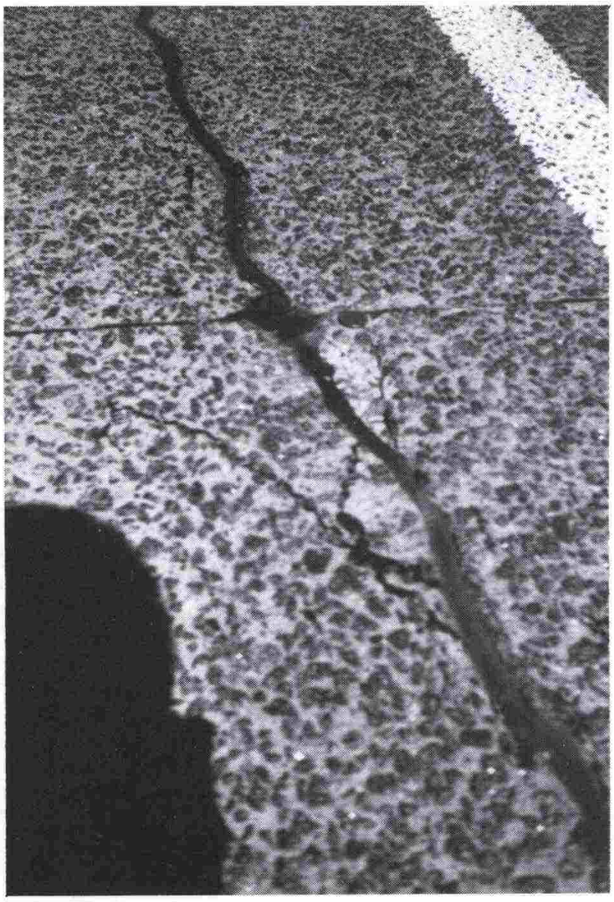
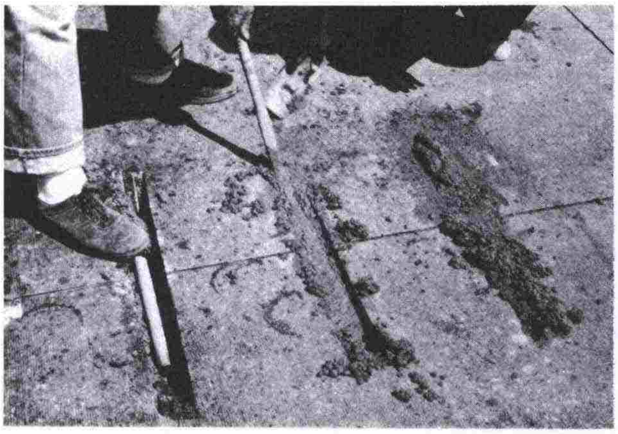


FIGURE B2-8. A repaired wild joint, spalling on the slab (Denmark 1988)



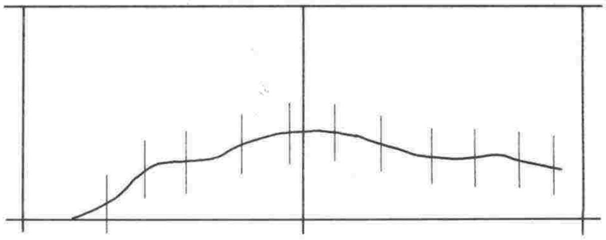


FIGURE B2-9. Dowel bars can be placed into a badly cracked slab to restore the load-transfer capacity /11/

FIGURE B2-10. Breaking out concrete within a thin bonded joint arris repair using a single headed scabbling tool

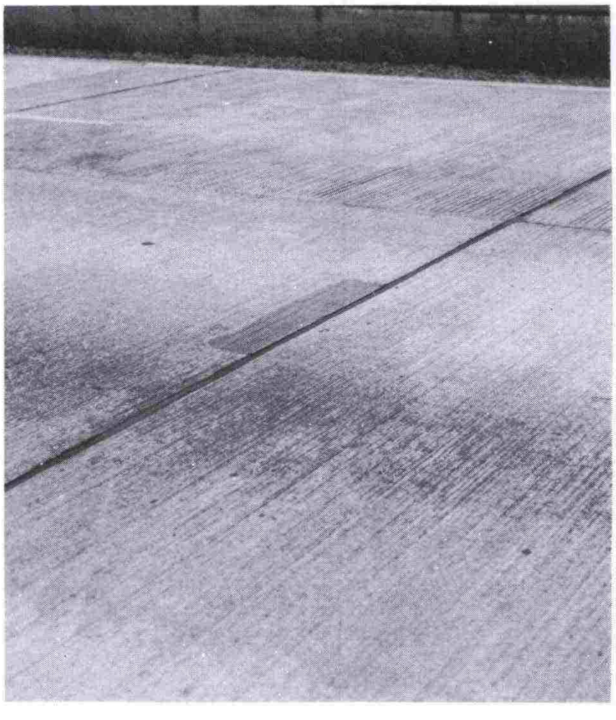


FIGURE B2-11. A succesful patch /1/

- The area to be renewed is marked out according to the following principles. A transverse repair must cover the total width of the lane, the length must be at least 1 m. The length of the remaining part of the old slab must be at least 1,5 m. Also a longitudinal repair is made along the total length of the slab, the width must be at least 1,0 m. A full-depth repair can also be made in the corner of the slab in which case the side length of the repair area can be 0,5-1,0 m. If the above margin requirements cannot be met, the entire slab will be replaced.

- The slab part to be renewed is sawn off at every side by a 20 mm wide diamond sawing. A wide saw-cut is necessary to be able to remove the slab part without breaking the edges.

- The old slab part is hoisted with a crane avoiding breaking the remaining slab edges and and with the minimum amount of damage to the subbase.

- Dowel bars are horizontally drilled (Figure B2-12, Figure B2-13) in new transverse joints.

- The dowel bars are grouted with epoxy into the old slab, the bar ends in the new slab are spread with bitumen.

- Concrete surfaces are cleaned with compressed air, temporary joint fillers are inserted and the base is made.

- Casting of the new slab part is undertaken in a traditional manner, particular care is taken to ensure good compaction, curing and evenness.

- The temporary joint fillers are removed, new joints are treated and finished like normal shrinkage joints.

Full-depth repairs cannot be opened to traffic until the concrete strength is at least 30 MN/m^2. If super-plastizised concrete is used, this means a traffic blockade of about 24 hours. If needed, strengthening can be accelerated with additives so that the repaired area can be opened to traffic in a few hours after the casting (fast track). Full-depth repairs are carried out in temperatures of 10 - 20°C in spring when there are no compressive stresses caused by heat.

B 235 Injection of slabs from underneath

When deflection of slabs under a traffic load are measured in damage evaluation, transverse joints with a slab rocking of > 3 mm can be located; such deflection can result in cracking or stepping of the slab. These kinds of slabs can be strengthened by injecting cement grout under the slab before the slab has damaged (Figure B2-14). Holes are drilled in the slab part to be injected, a square of approximately 1 m x 1 m, the nozzles are locked pressure-tightly to the pavement, the slab base is dried by blowing compressed air through the nozzles, the injection is undertaken with cement grout, the consistency and grain size of which depends on the void content under the slab. Injection must be stopped immediately when consumption of cement grout is decreasing in order to prevent pressure from lifting slabs in an undesirable manner. The holes are patched after the injection and the road can be openend to traffic when the samples taken during the process have given a strength value of 5 MN/m^2, which means in a couple of hours. If plastic grouts are used the road can be opened to traffic in an hour after the injection.

FIGURE B2-12. Drilling holes for dowel bars in a full depth repair

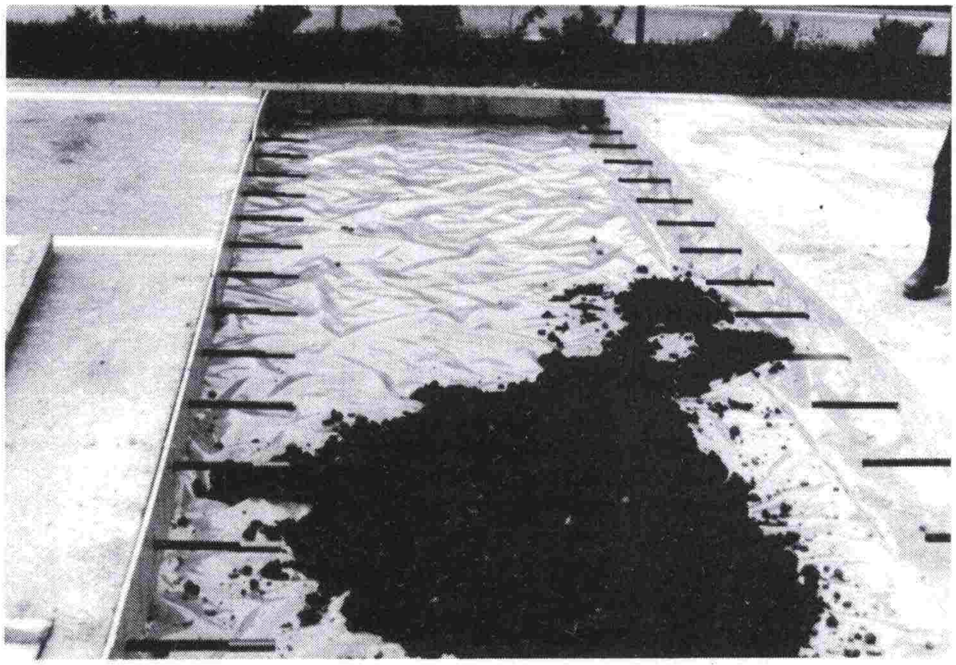


FIGURE B2-13. A full depth repair: dowel bars placed, the base compacted, casting with super-plastizised concrete can be started

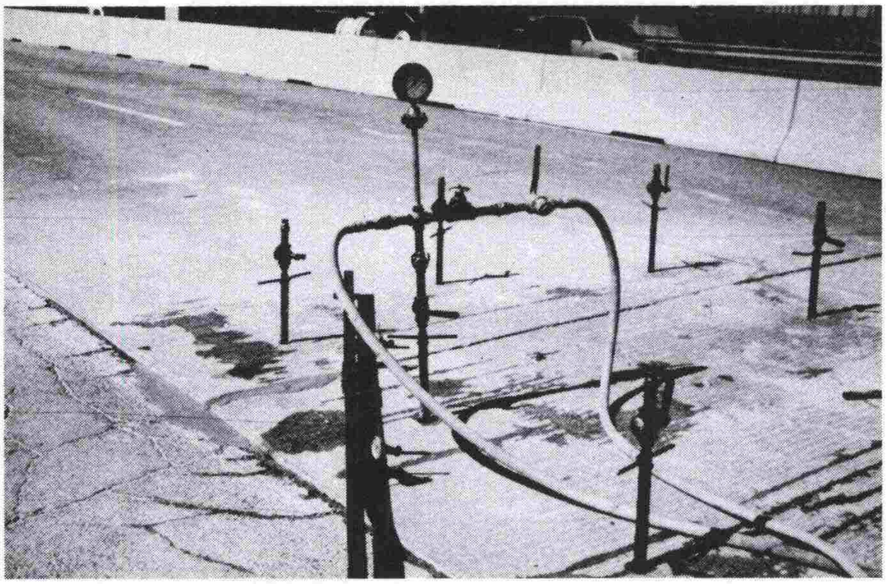


FIGURE B2-14. Injection of a rocking slab from underneath /17/

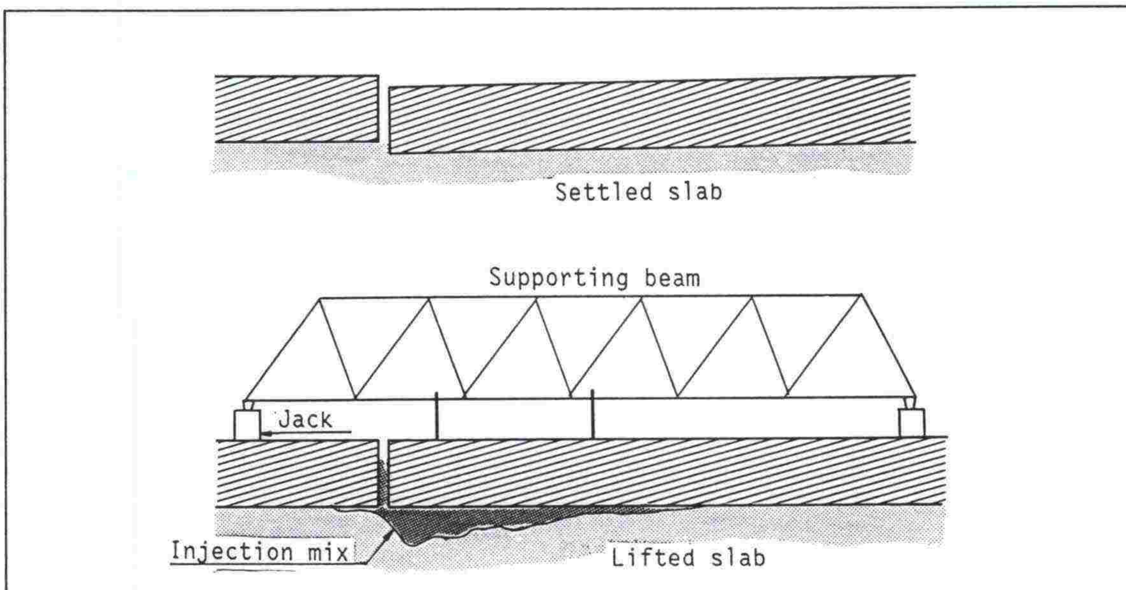


FIGURE B2-15. Slab lifting by means of supporting beams

B 236 Slab lifting

For various causes a depressed slab or a group of slabs can be lifted with special arrangements to the right level again. In principle, the lifting can be undertaken in two ways. Trusses can be installed over the slab to be lifted (Figure B2-15) and the lifting is carried out by means of jacks exactly to the required level; when the slabs are supported by the jacks an injection of the base is executed. Another way is to drill injection holes through the depressed slab group (Figure B2-16) and to carry out the lifting with the pressure of the grout compressed under the slab. In addition to injection equipment no other special equipment is needed in this method and the height of the rise can be controlled by simple measuring wires. This method is suitable for repair of even greater settlements but the lifting must be carried out very slowly and systematically, in order to avoid cracks on the slabs. Injection grout must be coarser grained and less liquid than that used in the injection described in the previous chapter.

B 237 Milling

Milling is often a profitable method of repairing concrete pavements. In principle, there are three different milling methods:/10/

1) Grooving

Transverse (sometimes also longitudinal) grooves are milled on a slippery pavement to improve friction. Grooves are milled as a diamond sawing for example at distances of 19 mm (Figure B2-17). If the intention is to improve only the drainage of the road surface, grooves can be less densely milled but then they are deeper (see Figure B5-15). Grooving is always carried out as a restricted local operation.

2) Grinding

The unevenness of the road surface is removed by diamond sawing. A special machine resembling a big motor grader (Figure B2-18) has diamond wheels side by side on two axles and the diamond wheels saw grooves densely on the pavement. Concrete between the grooves will break and it will be removed from the road. The intention of the grooving is to improve friction qualities, whereas the aim of the grinding is to remove material and to make an even road surface. A mild and dense longitudinal grooving (Figure B2-17) is left as a final surface. Grinding does not damage joints.

When grinding is used for example in guarantee repairs to improve evenness of new pavements only the occurring high areas are ground. When an old pavement is repaired by grinding, the pavement is treated at its total length. Grinding is often the last phase of a rehabilitation project, other pavement damage is first repaired locally according to the previous points.

Diamond grinding has been used to improve evenness of concrete pavements in the United States since 1965. The method has been developed into a reliable routine method which meets or even exceeds the evenness values of a new pavement (Figure B2-19 a, b).

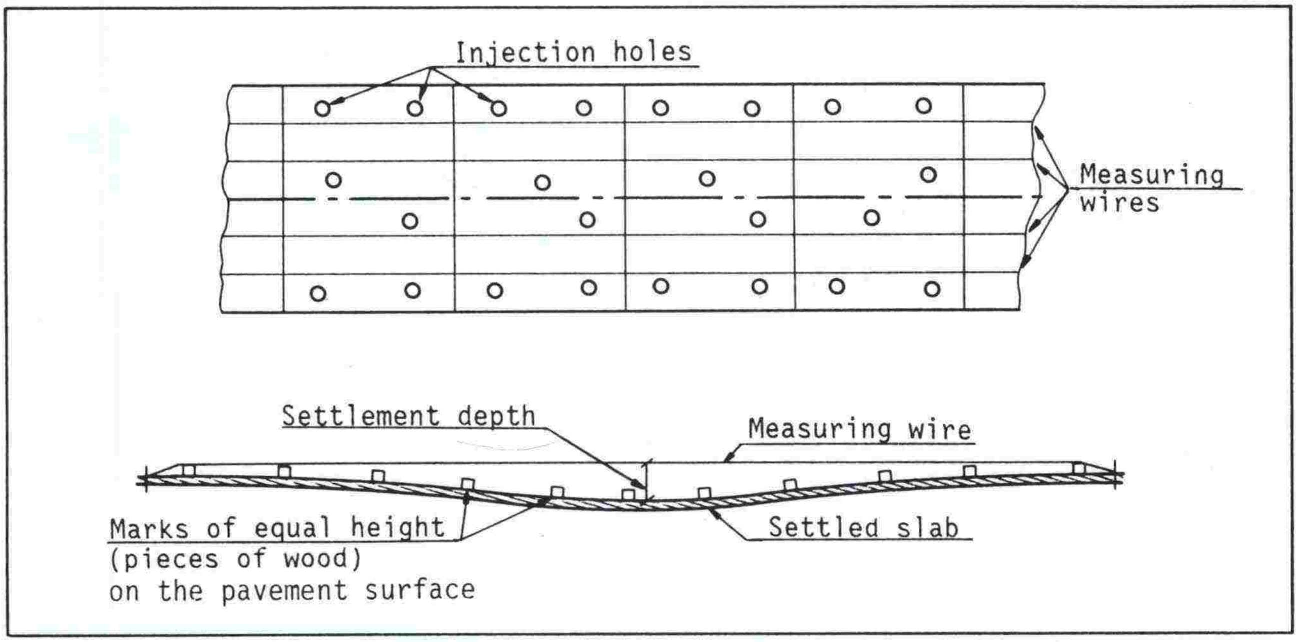


FIGURE B2-16. Slab lifting at a settlement /11/

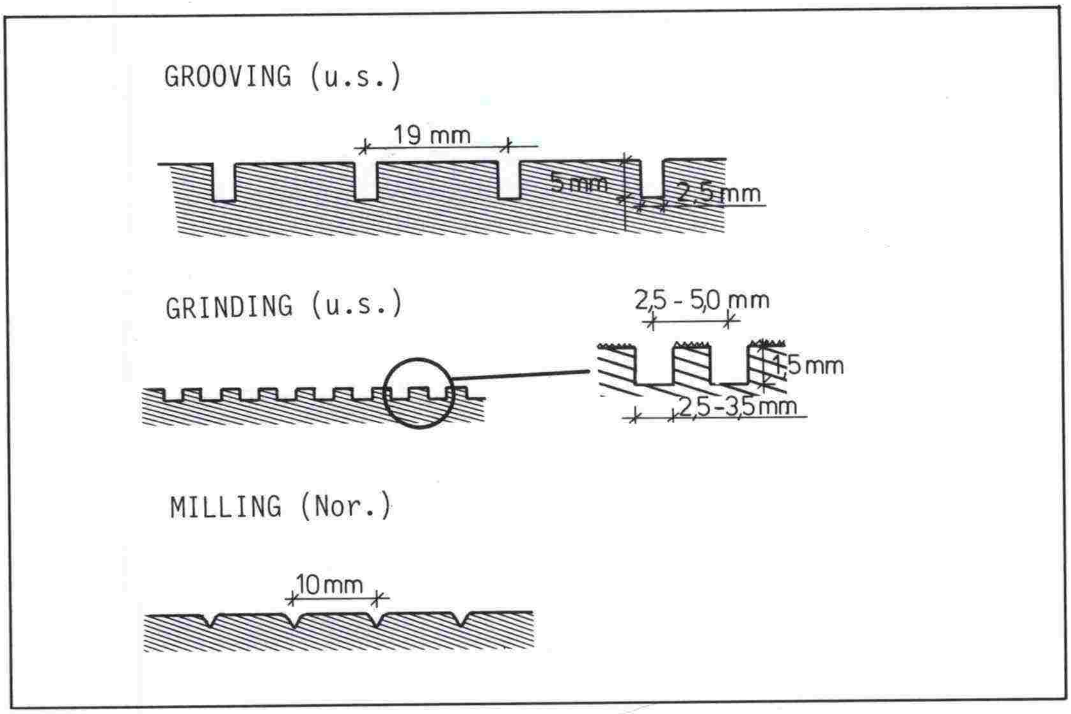


FIGURE B2-17. Typical grooving procedures /10/

FIGURE B2-18 a. A diamond fraise working; diamond wheels on two axles at the rear of the machine /6/

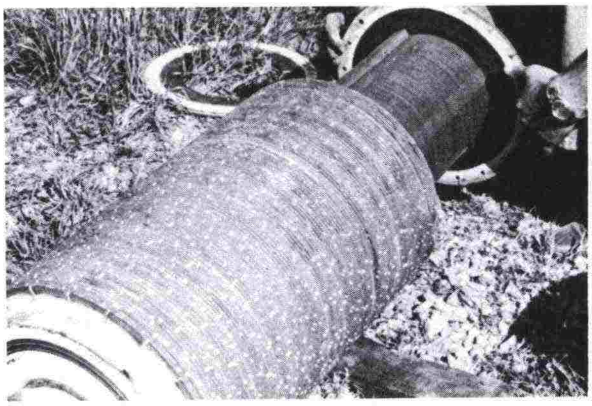


FIGURE B2-18 b. A close shot of the diamond wheels /6/

3) Cold-milling

Cold-milling is commonly used to remove old pavement material in rehabilitation of both asphalt and concrete pavements. Milling is undertaken by means of hard metal sharp-pointed cutters attached to the milling drum, Figure B2-20 a, b. Different kinds of cutters are used for milling of asphalt or concrete, otherwise the work is performed in the same way. Cold-milling of concrete was originally intended to clean or reduce the thickness of the old pavement before casting a new bonded overlay. Cold-milling is still forbidden in the United States if the milled surface will be trafficked, because cold-milling is considered to damage joints of the concrete pavement and to considerably increase noise and wear of tyres.

Instead, since 1978 Norway has tested and applied the latest cold-milling methods to improve evenness of concrete pavements /54/. According to the Norwegian experience a cutter distance of 10 mm and a low speed have a smaller effect on joints and result in a lower grooving than a longer cutter distance and a higher speed (Figure B2-17). Also milling machines have been developed in Norway and cold-milling is ordinarily applied to rehabilitation of rutted concrete pavements caused by studded tyre traffic.

FIGURE B2-19 a. Concrete pavement repaired by diamond grinding in Alabama in the United States

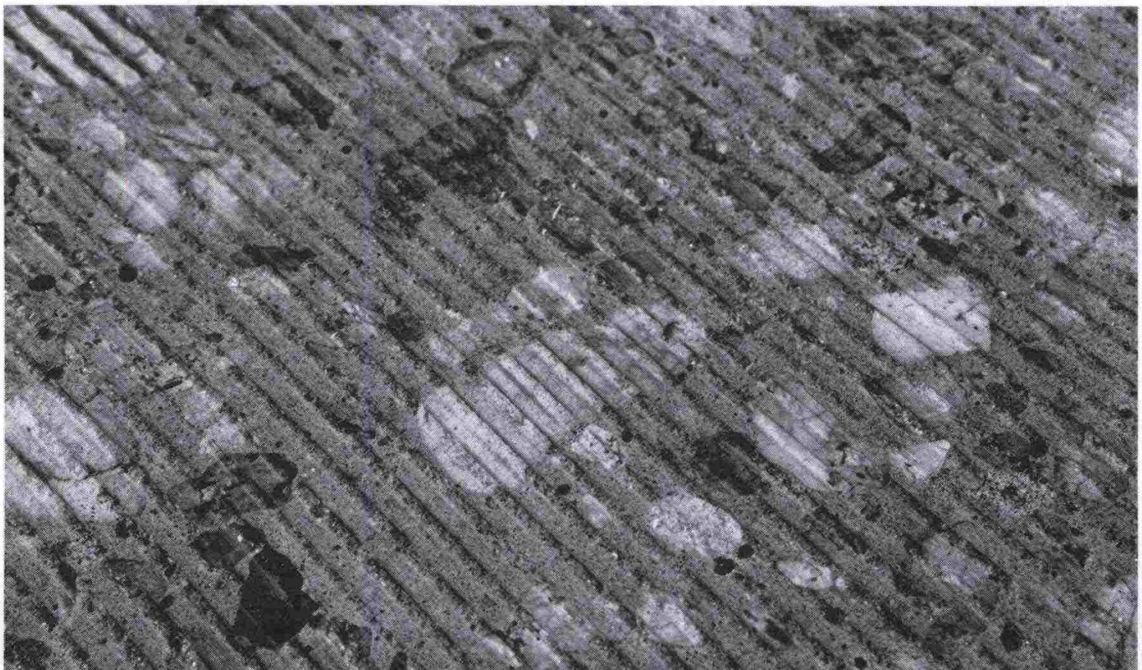


FIGURE B2-19 b. A detail of the diamond-grinded concrete pavement

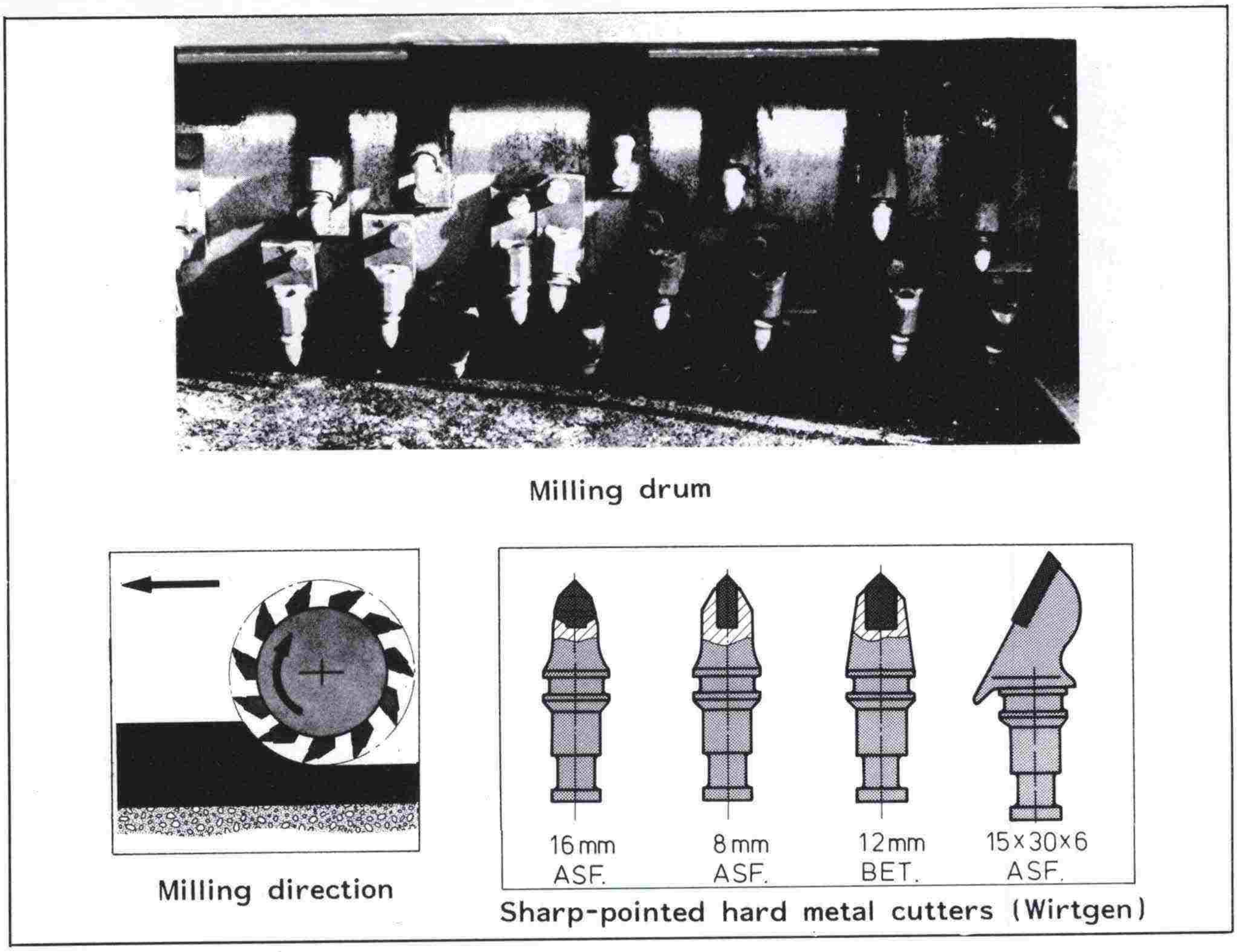


FIGURE B2-20 a. Milling drum and sharp-pointed hard metal cutters in cold milling

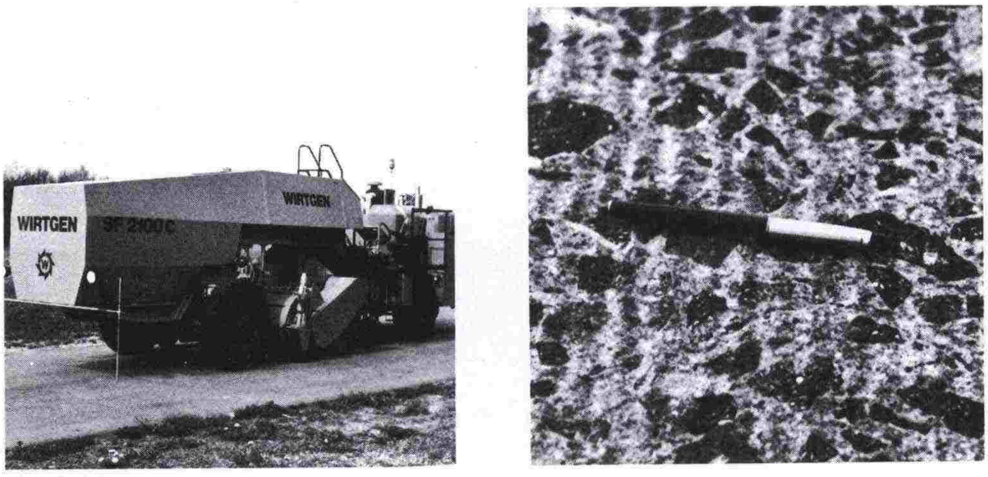


FIGURE B2-20 b. Milling machine and a cold-milled area on a concrete pavement

B 238 Surface treatments and thin overlays

Friction, evenness and homogeneity of the surface can be improved not only by milling but also by different surface treatments and overlays. The purpose of a treatment may be

- impregnation of the pavement concrete
- improvement of the water tightness of the surface
- adhering of chippings to the surface to improve friction

Usually surface treatment material is uniformly spread on the entire road, but this can also be made locally for example at a crack or at an open road surface area. Plastic- or oil-based liquid materials are used as surface treatment materials; every material has its own dosage and working technique. The surface must be clean and dry to achieve a succesful outcome. Surface treatments to improve friction are undertaken with epoxy or bitumen. Epoxy has often been abandoned because the pavement may become even more slippery when the chippings wear out.

An open area can be sealed, a local uneven area can be repaired or a new even surface can be achieved with a thin overlay. When it is a question of a local repair the surface is milled or grooved and then cleaned with compressed air. The mix is spread with hand equipment and finished to the level of the pavement surface (Figure B2-21). Cement or epoxy grout is used as a mix. When using cement grout the thickness of the grout must be at least 10 mm even on the edges, also 'zeroing' can be made with epoxy grout. Epoxy grout gives a durable and tight surface but the work must be done very carefully and in dry conditions. A thin epoxy overlay should not be greater than 1 m x 1 m, otherwise cracking and peeling will increasingly occur.

A thin overlay covering the entire pavement is made on a milled concrete surface. If concrete is used as a mix (with or without steel fibres), the minimum thickness of the overlay is 35-50 mm, when using plastic additives or asphalt a minimum thickness of 20 mm can be reached. Great interest is shown in these very thin overlays all over the world. So far, only tests have been made and the experiences have been variable- and the price often high. An Austrian investigation on the mix design of thin overlays is presented in Figure

B2-22. Bonded overlays are also referred to in Chapters B 24 and B 271.

B 239 Other rehabilitation methods

The above described repair and rehabilitation methods concern the pavement slab itself. The age of the pavement can also be extended by indirect measures, such as:

- by building a concrete shoulder attached to the old carriageway slab
- by improving drainage by means of side subdrains

Old concrete pavements are usually not wider than the carriageway, shoulders are usually paved with asphalt. Edge stresses of carriageway slabs can essentially be decreased - according to some calculations even by 30 % - by building concrete shoulders and by binding them to the carriageway slab by means of tie bars. If traffic is heavy and the pavement is in a good condition but measurements show deflections at joints, construction of concrete shoulders should be considered.

FIGURE B2-21. A thin epoxy overlay as a repair method of an open pavement surface

Binder	Cement	Cement+acrylic plastic
Minimum thickness	35 mm	20 mm
Aggregate - fine aggregate - rough aggregate	 45 % 0-4 mm 55 % 8-12 mm	 38 % 0-2 mm 62 % 4-8 mm
Portland-cement	455 kg/m^3	540 kg/m^3
Acrylic plastic liquid	-	100 kg/m^3
Air entraining agent	0,1 % of the amount of cement	0,06 % of the amount of cement
Super-plastiziser	0,2 % of the amount of cement	0,1 % of the amount of cement
Water/cement ratio	0,35	0,35
Shrinkage (48 days)	0,08 %	0,05 %
Flexural strength (28 days)	6,9 MN/m^3	6,0 MN/m^2
Compressive strength (28 days)	46 MN/m^2	19 MN/m^2

FIGURE B2-22. Example of bonded overlays according to an Austrian investigation /36/

It is an acknowledged fact that inadequate drainage has been the most important single environmental factor resulting in deterioration of old concrete pavements. On the other hand, there are differing opinions of the efficiency of side subdraines built afterwards. Anyhow, subdrains are more and more often included in new rahabilitation projects. (see Figure B4-31). The idea of subdrains is to quickly remove rainwaters from the carriageway and from the road structure in all conditions and to avoid detrimental effects of water on the load-bearing capacity and the durability of the base.

B 24 REPAIR OF RUTS

Because repair of ruts is a typical rehabilitation problem in the Finnish conditions it will separately be dealt with here.

There are not much experience of the repair of ruts on concrete pavements. However, the existing concrete pavements started to rut alarmingly all over the world in the beginning of the studded tyre age in the 1960s. The lack of rut repairing methods was one of the main reasons for the prohibition of studded tyres in many countries. Badly rutted concrete pavements were paved with an asphalt layer which was enough when studded tyres were not used any more. Only Austria and Norway have done wider research on rut repairing methods and costs. In principle there are five different rut repairing methods (B2-23):

a) Surface treatment

Incipient rutting can be retarded by adhering a firm durable chipping at the rut using bitumen, rubberized bitumen or resin mortars as a binding agent. The method is used in rehabilitation of slippery ruts among others in France and England. It was established in the tests made in Norway that the method could not withstand the heavy studded tyre traffic long enough.

b) Filling of ruts with mixes

Filling a rut for example with fine-grained rubberized asphalt mix is an applicable method, if the "colour defect" is not considered an inconvenience. This method has been used in Austria (Figures B2-45, B2-46). This is a short-term method if the rut filling is undertaken with asphalt. The durability can- to some extent - be improved by surface treatment. Use of other binders - epoxy, resin, sulphur- has been tested but they have proved to be expensive and difficult in practice. Laboratory precision in different working phases is required to bring about a good adhesion in the "zeroing" of the rut edges. Until now the method has not suited for a more wide-spread application in rut patching, but it is practical as a local patching method.

c) Grooving and filling of ruts with concrete mix

A method for rutting repairs has been developed in Austria; there a 3,5 cm deep hole is milled at the rut, cleaned out by compressed air and then filled with super-plastizised concrete (Figure B2-24) (see also Chapter B 238). If well performed this method has every qualification to succeed and it is well-suited also for wide-scale patching. According to a Norwegian test the method was regarded as expensive and inconvenient and as too complex in practice. As good a surface regularity as that of the rest of the carriageway was not acchieved in the test; the work itself succeeded and the repairs withstood traffic satisfactorily.

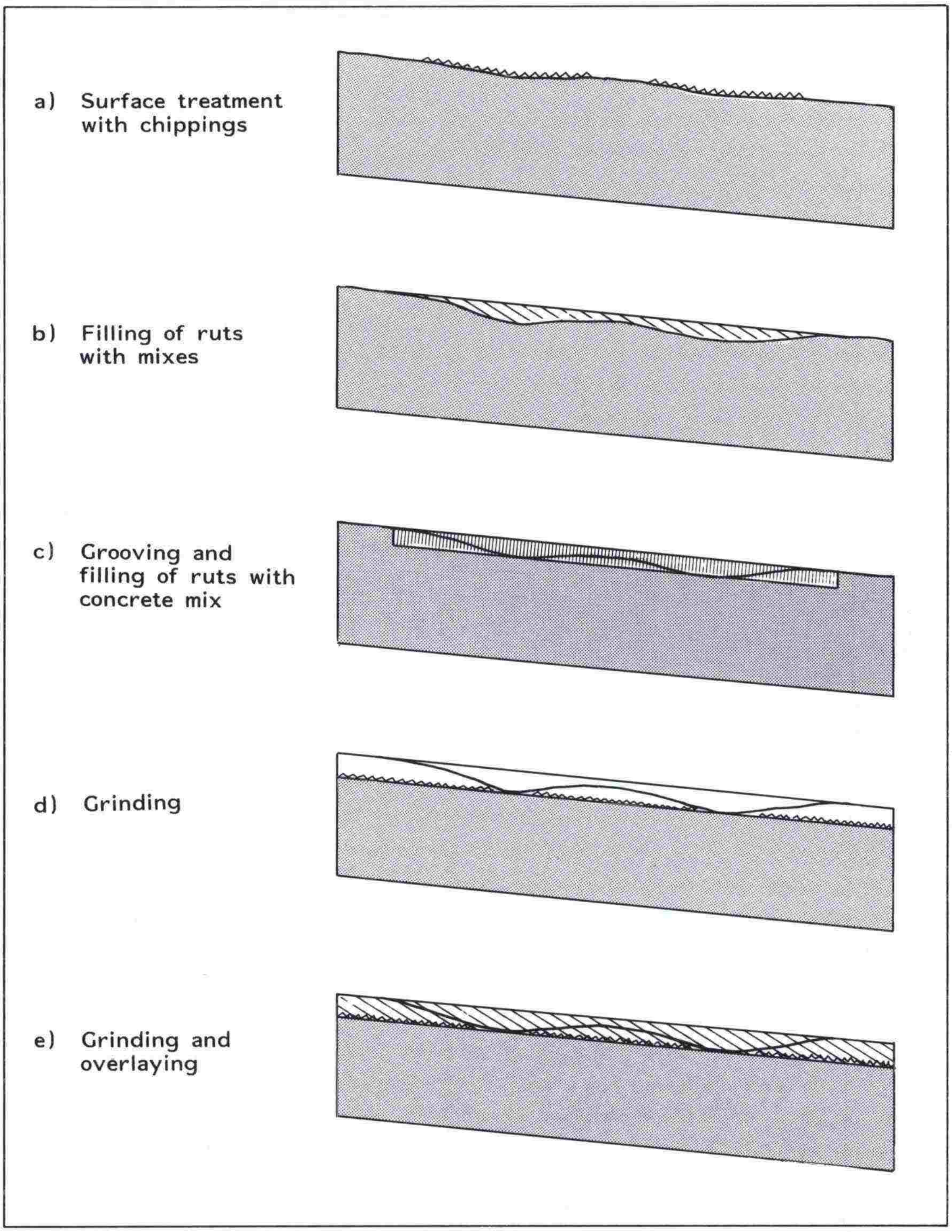


FIGURE B2-23. Repair methods of ruts of concrete pavements

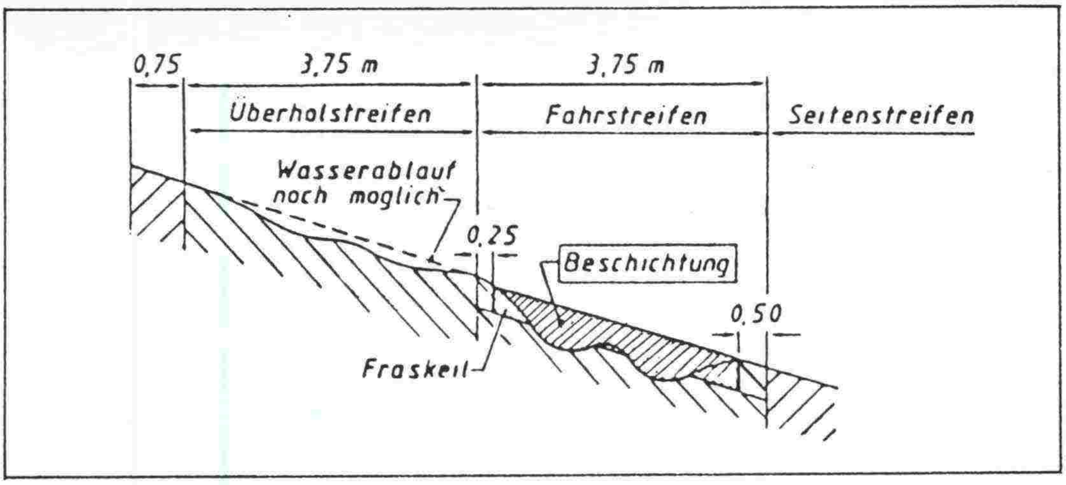


FIGURE B2-24. Repair of ruts with concrete mix according to an Austrian research /36/

d) Milling and grinding

Clearly the most interesting method of rutting repairs is the milling of an unworn pavement section to the level of the rut root. Precise and efficient diamond and cold milling methods and equipment have been developed in the United States; their adaption for levelling of ruts is a result of the latest, primarily Norwegian, development work. The first wider milling tests were carried out on the Main Road E18 in Tonsberg Norway by Danish cold milling equipment in 1979 and 1980. Later a special fraise attached to the motor grader was developed in Norway; bridge pavements, among others, were milled with it. Encouraged by the experience the road authorities executed a 4 km long milling project with very good results in 1982. Two big American and German cold milling machines were used in this work and they were later bought to Norway. Based on good experiences milling is a routine method in Norway and the technical problems so far connected with rutting repairs are solved by it. In 1983 the milling method was tested in Finland on the Ring Road III built in 1970. A 23 ton machinery (Roconeco Ingersoll Rand MW-6520 XP) was used; the working width of the milling drum is 2,0 m. The milling depth of concrete was approximately 14 mm and totally an area of 840 m^2 was processed. This Finnish test was also very succesful and taught much both in working methods and applicability.

e) Milling and overlaying

E.g. the following reasons may result in overlaying a milled surface:

- it is not possible to resort to diamond milling, or cold milling does not give satisfactory results

- the milled slab is so thin that it won't withstand repeated traffic loads

- the purpose is to obtain an especially durable surface (steel fabrics, epoxy etc.)

Application of an overlay is made according to the principles of Chapter B 238. The method has been used to some extent in rehabilitation of bridge pavements. The method is unreasonably expensive merely for repairs of ruts, but it may be considered in case of also other repairing objectives.

Of the above methods the milling method is the most widely developed and the most applicable for widespread use. In a careful design of pavement slabs rutting is taken into account as an additional thickness of the slab. Thus the millings can be performed without endangering the constructional durability of the slab.

B 25 DESIGN OF REPAIR AND REHABILITATION MEASURES

B 251 Selection and timing of the measures

The above repair and rehabilitation methods have been described in a general sense without a reference to the repair methods of the different damage types especially shown in Figures B2-1, B2-2 and B2-3. For each project the most proper methods are to be selected of the methods described or of their combination.

In addition to the actions also the time of the repair or rehabilitation operation must be decided in connection with the design. Regular and systematical damage evaluation is of good help. New damage demanding quick measures is visible to the eye and their worsening or constancy can be estimated by comparing the collected information with the earlier data. Thanks to regular damage evaluation the condition can be followed up in order to avoid negligences and also over-sized repairing measures. Ready-made descriptions of the degree of difficulty and of various standby limits are available. The evenness and service level of the road are also good indicators. An interdependence of rehabilitation needs and service level index according to an American investigation are shown in Figure B2-25./25/ In practice spalling and cracking are attended to during an annual maintenace operation, rutting and resealing of joints are taken care of at certain annual intervals and a wider maintenance is considered at the end of the service life of the pavement (20-30 years).

B 252 Profitability of the measures

On the basis of the above survey a technically reliable repairing method can be found for every occurring damage. The preliminary design must also estimate how drastic measures are considered profitable on the whole. When defining the profitability of a rehabilitation operation not only the costs of the rehabilitation are important but also how much the service life of the pavement can be extended (Figure B2-26). As far as a single rehabilitation operation is concerned the service life is the decisive factor.

When service life is examined, the service life of a single operation should be separated from the increase in a service life of an entire pavement sector achieved by rehabilitation operations. It is important- but difficult - to estimate both the factors and it requires good realization data and wide experience. Certain service life figures typical of some rehabilitation measures are collected to Figure B2-27.
Figure B2-28 shows an American estimation of the influence of a wide rehabilitation operation on the service life of the pavement.

The investment costs or repair works are to be estimated action by action on the basis of local price data. Certain foreign unit prices are shown in Figure B2-29. They are adaptable only in the conditions of the country in question, but they give an idea of the size range of repair and rehabilitation costs.

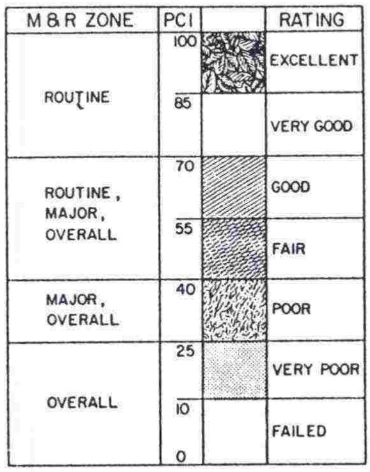


FIGURE B2-25. Interdependence of rehabilitation needs and service level index according to an American investigation /25/

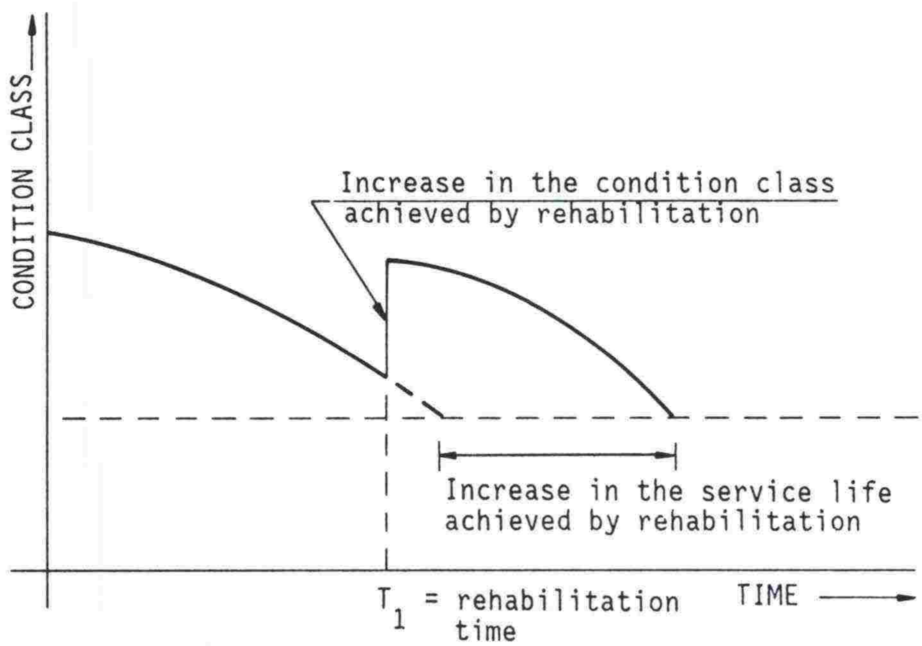


FIGURE B2-26. Increase in the service life thanks to rehabilitation measures /15/

When the investment costs and the increase in service life have been estimated an annual cost caused by the rehabilitation can be calculated and this can be used as a characteristic when comparing the profitability of different rehabilitation strategies. (Figure B1-23)

REHABILITATION MEASURES	PREREQUISITES	ESTIMATED SERVICE LIFE (years)
- full depth repair	a healthy slab	more than 10 years
- patching	a healthy slab	5 - 10 years
- injection from underneath	good drainage, joints in a good condition	4 - 8 years
- restoring the load-transf. capacity by means of dowel bars	good drainage, joints in a good condition	3 - 7 years
- construction of a concrete shoulder to diminish stresses at edges	injections, patching when needed	10 - 20 years
- milling	injections, patching made; drainage, joints in a good condition	5 - 15 years
- renewing of joint sealants	rubberized bitumen silicone	4 - 6 years 8 - 15 years

FIGURE B2-27. Evaluations of single rehabilitation measures according to an American investigation /15/

HEAVY TRAFFIC (vehicles/lane/day)	RAINFALL (mm/Y)	ESTIMATED INCREASE OF SERVICE LIFE
1. < 1500	< 500	15 - 20 years
2. > 1500	< 500	10 - 15 years
3. < 1500	> 500	10 - 15 years
4. > 1500	> 500	5 - 10 years

FIGURE B2-28. An increase of the service life achieved by a full-scale repair project according to an American investigation /15/ (Note! Group 3 corresponds to the Finnish conditions)

UNIT PRICES FOR REHABILITATION MEASURES, (Minnesota, USA, 1984, /29/)

Renewal of the transverse joint (cleaning, sawing, filling)	16,00 mk/m (1$ = 4,35 mk)
Repair of a crack (grooving, filling)	19,00 "
Renewal of the longitudinal joint (cleaning, sawing, filling)	11,50 "
Patching (grooving, cleaning, filling)	90,00 "
Full-depth repair (small)	100,00 "
Full-depth repair (wide)	195,00 "
Installation of dowel bars	34,00 mk/kpl
Sawing a new joint	15.50 mk/m
Replacement of pavement (removal of old pavement, rehabilitation of base, construction of new pavement)	275,00 mk/m2
A bound slab on top of the old pavement, 7,5 cm (including crack repairs)	119,00 mk/m2 (Dakota 1985)

UNIT PRICES FOR REHABILITATION MEASURES, (Norway, 1983, /55/)

Cold milling (depth 30 mm)	13,00 mk/m2 (the price rises 3,50 mk/m2 per each additional depth of 10 mm)

UNIT PRICES FOR REHABILITATION MEASURES, (USA, 1981, /32/)

Polymer concrete (for repairs)	min. 1300 mk/m3 up to 10000 mk/m3

FIGURE B2-29. Unit prices for rehabilitation measures (note! the prices depend on the local price level - these prices give only the size range)

B 26 PREVENTION OF DAMAGE

Damage on a pavement leads inevitably to repaving or heavy rehabilitation in the course of time. Although the service life of concrete pavements is long, the threshold of a reinvestment is so high that it is tempting and often even profitable to extend the service life even more and thus postpone greater investments. This object can be achieved by analyzing the design, construction and maintenance problems because a long service life will or won't be achieved depending on the decisions of these working phases.

1) Means of the design

Defects of old pavements and before all their causes give good knowledge of the design of new pavements. A wide investigation on the condition of pavements (COPES/9) recently carried out in the United States gives also statistical representativeness to the following conclusions:

- addition of the slab thickness
  reduces cracking, faulting and pumping and delays the decrease in the service level.

- shortening of slabs
  reduces faulting, joint damage and cracking and delays the decrease in the service level

- use of dowel bars and addition of bar thickness
  reduces faulting, improves the service level

- a bound base (subbase)
  reduces faulting and cracking

- improved drainage
  reduces pumping and the decrease in the service level

- a good joint filler (which prevents penetration of foreign materials into the joint)
  reduces spalling and cracking at the slab ends

These are well-known keyfactors in the design of concrete pavements. The American research only strengthens the opinion of their importance of the long-term durability. Thus it is evident that cutting down on construction costs (slab thickness, dowel bars, jointing compounds etc) is fatal for the long-term durability of the pavement, although it would seemingly improve the competitiveness of the concrete pavement. On the other hand, the service life can considerably be extended by expending more than usual on the structure.

2) Means of construction

The quality level of the construction work has a great effect on the damage speed of the pavement. Construction failures are the principal reason for the main part of the pavement defects. As to the long-term durability the following significant failures can be mentioned (see also Chapter B 154):

- a wrong position of dowel bars
- unhomogeneous mix
- a delayed sawing of joints
- a bad curing of fresh concrete
- wrong material choices (a weak or reactive aggregate)
- other reasons

The success of the construction work is not thus measured only by the evenness of the pavement but also by small rehabilitation costs and long-term durability.

3) Means of maintenance

The significance of annual rehabilitation measures have lately been emphasized all over the world. Road authorities generally regret that rehabilitation of concrete pavements of the previous generation have been neglected and that defiencies have progressed too far. An annual rehabilitation doesn't generally prevent development of damage but it contributes to restraining them and to maintaining the constructional durability and good service level as long as possible. The strategy of constant maintenance is wise economy also in the maintenance of concrete pavements.

B 27 RECONSTRUCTION OF CONCRETE PAVEMENTS

The economic service life of the concrete pavement comes to an end in due course and rehabilitation has to give way to reconstruction. Because the old pavement is attempted to be utilized with as great a terminal value as possible in connection with the reconstruction, the border line between rehabilitation and reconstruction is sometimes wavering. In this context the border line is determined by the fact whether the intended actions only will extend the service life or whether the durability of the renewed pavement equals that of a new pavement. In this report the attention has been on repair and rehabilitation problems. Only the most common characteristics of reconstruction methods are dealt with in the following.

B 271 Bonded overlays

If a concrete pavement is in a good condition but its thickness is short-dimensioned to the actual heavy traffic flow, thickness can be added by casting a new thin slab on the surface of the old one and by binding the slabs together (bonded overlay, Figure B2-30). The old slab surface is milled and cleaned and cement grout is spread on it just before the casting of the new surface slab. The new slab won't be reinforced (no dowel bars); the joints are sawn through the new slab at the joints of the old slab. The thickness of the slab is generally under 10 cm. A reliable bond between the slabs is a qualification for success.

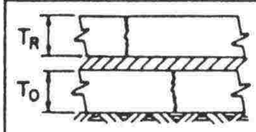
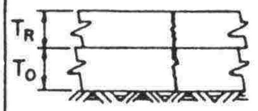
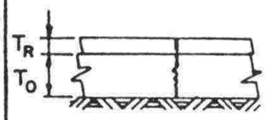
If there are more defects on the old slab and its remaining service life would be shorter than described above, a thicker slab can be cast without milling (partially or direct bonded overlay), Figure B2-30. The damage of the old slab is carefully repaired, the surface is brushed and a new slab is cast directly on the old slab. The slabs are regarded to partly function together which is taken into account in the thickness dimensioning.

A summary and applicability of bonded overlays are presented in Figure B2-30.

B 272 Bonded overlays with asphalt

When an old slab is in a good condition otherwise but the pavement has lost its evenness, it can be paved with asphalt using a thin (4-5 cm) asphalt overlay. The damage of the old pavement is carefully repaired with asphalt mix. Joints can be sawn and filled on the asphalt pavement at the joints of the old slab or no joints are made; reflection cracking can be reduced by spreading a reinforcing fabric at the joints before paving with asphalt. Reflection cracking is to be expected. The stability of the asphalt mix must be good because a thin asphalt layer between a rigid slab and heavy traffic is prone to great stresses and it ruts easily.

CONCRETE OVERLAYS ON CONCRETE PAVEMENT

	UNBONDED OR SEPARATED OVERLAY	PARTIALLY BONDED OR DIRECT OVERLAY	BONDED OR MONOLITHIC OVERLAY
TYPE OF OVERLAY			
PROCEDURE	CLEAN SURFACE DEBRIS AND EXCESS JOINT SEAL PLACE SEPARATION COURSE-PLACE OVERLAY CONCRETE.	CLEAN SURFACE DEBRIS AND EXCESS JOINT SEAL AND REMOVE EXCESSIVE OIL AND RUBBER-PLACE OVER-LAY CONCRETE	SCARIFY ALL LOOSE CON-CRETE, CLEAN JOINTS, CLEAN AND ACID ETCH SURFACE - PLACE BONDING GROUT AND OVERLAY CONCRETE.
MATCHING OF JOINTS IN OVER-LAY & PAVEMENT — LOCATION	NOT NECESSARY	REQUIRED	REQUIRED
MATCHING OF JOINTS IN OVER-LAY & PAVEMENT — TYPE	NOT NECESSARY	NOT NECESSARY	REQUIRED
REFLECTION OF UNDERLYING CRACKS TO BE EXPECTED	NOT NORMALLY	USUALLY	YES
REQUIREMENT FOR STEEL REINFORCE-MENT	REQUIREMENT IS INDEPENDENT OF THE STEEL IN EXISTING PAVEMENT OR CONDITION OF EXISTING PAVEMENT.	REQUIREMENT IS INDEPENDENT OF THE STEEL IN EXISTING PAVEMENT. STEEL MAY BE USED TO CONTROL CRACKING WHICH MAY BE CAUSED BY LIMITED NON-STRUCTURAL DEFECTS IN PAVEMENT.	NORMALLY NOT USED IN THIN OVERLAYS. IN THICKER OVERLAY STEEL MAY BE USED TO SUPPLEMENT STEEL IN EXISTING PAVEMENT.
FORMULA FOR COMPUTING THICKNESS OF OVERLAY (T_r) NOTE: T IS THE THICKNESS OF MONOLITHIC PAVEMENT REQUIRED FOR THE DESIGN LOAD ON THE EXISTING SUPPORT C IS A STRUCTURAL CONDI-TION FACTOR T_R SHOULD BE BASED ON THE FLEXURAL STRENGTH OF	$T_r = \sqrt{T^2 - CT_0^2}$ OVERLAY CONCRETE	$T_r = \sqrt[1.4]{T^{1.4} - CT_0^{1.4}}$ OVERLAY CONCRETE	$T_r = T - T_0$ NOTE: THE ABILITY OF THE OVERLAID SLAB TO TRANSFER LOAD AT THE JOINTS SHOULD BE ASSESSED SEPARATELY EXISTING CONCRETE
MINIMUM THICKNESS	6"	5"	1"
APPLICABILITY OF VARIOUS OVERLAY TYPES — STRUCTURAL CONDITION OF EXISTING PAVE-MENT — NO STRUCTURAL DEFECTS C = 1.0*	YES	YES	YES
APPLICABILITY OF VARIOUS OVERLAY TYPES — STRUCTURAL CONDITION OF EXISTING PAVE-MENT — LIMITED STRUCT. DEFECTS C = 0.75*	YES	ONLY IF DEFECTS CAN BE REPAIRED	ONLY IF DEFECTS CAN BE REPAIRED
APPLICABILITY OF VARIOUS OVERLAY TYPES — STRUCTURAL CONDITION OF EXISTING PAVE-MENT — SEVERE STRUCT. DEFECTS C = 0.35*	YES	NO	NO
APPLICABILITY OF VARIOUS OVERLAY TYPES — SURFACE CRACKS, SCALING, SPALL-ING AND SHRINK-AGE CRACKS — NEGLIGIBLE	YES	YES	YES
APPLICABILITY OF VARIOUS OVERLAY TYPES — SURFACE CRACKS, SCALING, SPALL-ING AND SHRINK-AGE CRACKS — LIMITED	YES	YES	YES
APPLICABILITY OF VARIOUS OVERLAY TYPES — SURFACE CRACKS, SCALING, SPALL-ING AND SHRINK-AGE CRACKS — EXTENSIVE	YES	NO	YES

* C VALUES APPLY TO STRUCTURAL CONDITION ONLY, AND SHOULD NOT BE INFLUENCED BY SURFACE DEFECTS.

FIGURE B2-30. Summary of concrete overlays on concrete pavement /5/

An asphalt pavement as a reconstruction method is always a temporary solution. The concrete pavement underneath will age and damage in due course and rehabilitation measures are needed in a fashion of an ordinary concrete pavement.

An asphalt-paved concrete slab is - when duly jointed - also a structural alternative of a new road among others in Canada, in the United States /22/ and England.

B 273 Unbonded overlays

When the old pavement is badly cracked it can no more be used as a part of the new pavement or as a base. In such a case the old pavement with its damage will be left as a subbase under an asphalt or some other separating layer and the new concrete pavement will be dimensioned fully independently. Also joints are placed independent of the old pavement, Figure B2-30.

If an intermediate asphalt layer won't be built or the behaviour of the old concrete pavement is doubtful the old pavement can be broken into smaller pieces (1-2 m^2) by special hammers and the pieces can be compacted by very heavy rollers (about 30 t) ('crack and seat' method). Then a new slab is cast without an intermediate asphalt layer. The method is widely used in the United States - also when the new pavement is of asphalt. The method was also used on the old concrete roads in Malmöhuslän in Sweden.

B 274 Recycling

When an old concrete pavement is not needed to improve the load-bearing capacity or it won't be left as such as a base, it is broken by mechanical vibrating hammers, hauled away and crushed to be used as pavement material either to a new concrete pavement or to a cement-treated subbase. The reinforcement is separated either on site or in a crushing plant. According to the American experience crushed concrete is well-suited as aggregate for a new concrete pavement /11/. Addition of sand is recommended only to improve workability. In Europe the old concrete pavement is generally returned crushed to the site as raw-material of cement-treated layers and traditional aggregates are used for a new pavement slab.

B 275 White-topping inlays and overlays

A heavily trafficked asphalt-paved road, where rutting is a problem, can be reconstructed into a concrete-paved road. This can be made either directly on the old asphalt pavement (white-topping overlay) or by milling a space for a concrete layer on the old asphalt (white-topping inlay), Figure B2-31. This means construction of a new concrete pavement in every respect; the old asphalt layer forms a good, load-bearing base, but doesn't essentially decrease the thickness of the concrete pavement. The decrease can be as much as 10 % at its best compared with the construction of a new traditional pavement. Quite a different matter is if the "white-topping" is reinforced or if it is made of steel-fibre concrete or of other special concrete. Then thickness has considerably been reduced in some tests. There is not enough proof of the economy or durability of these special solutions, at least not for the present.

White-topping solutions have been executed both in the United States and Europe and they are considered more and more often a solution for load-bearing and stabilization problems of heavily trafficked urban roads in European metropolitans. The same principle is carried out in the construction of the Austrian motorways so that the motorways are paved with asphalt in the first construction phase, but later a concrete slab will be built as a final pavement.

B 28 FOREIGN EXPERIENCES OF THE DESIGN AND REALIZATION OF REPAIR AND REHABILITATION WORKS

B 281 Experiences from the United States /6,61,62/

It has been estimated in the United States that 15000 km (about 25%) of the Interstate motorway network is in need of immediate rehabilitation and the need is constantly increasing. The main road network built in the 1950s and 1960s has started to deteriorate dramatically due to age and to greater increase in traffic than anticipated. About half of this road network has been paved with concrete. The situation characterized as an emergency led to many strong actions on the federal level since the end of 1970s:

- A so called 4R-programme was started (Interstate Resurficing - Restoring - Rehabilitating-Reconstructing). The aim was to save the Interstate network by securing financing and technical conditions for rehabilitation. This programme is still some kind of a frame-work for all other efforts started.

- Federal financing activites for rehabilitation of the Interstate network has been quadrupled during the five years 1982-1987.

- Remarkable development projects to develop rehabilation technique have been executed in co-operation between road authorities, contractors, manufacturers and researchers.

- A wide international SHRP research programme (Strategic Highway Research Program) was planned to investigate factors effecting the condition and service life of the road network and to improve instructions and technique on its basis. The project was started as a 5-year research work in 1987. Thorough laboratory in vestigations and also wide field evaluations and construct ion of test roads are included. Also experts from abroad take part in the project, Finland is participating in the co-operation with the rest of Scandinavia.

The 4R-project as a whole includes of course both asphalt and concrete roads. When half of the Interstate-roads are concrete roads, it is evident that the maintenance problems of concrete roads are an important part of this research and development. Detailed new instructions on design and realization of maintenance of concrete pavements have already been published on the federal level in the 1980s. Up-to-date instructions on rehabilitation of concrete pavements are already included in the road standards in many states.

In practice the first R (= resurficing) of the 4R-project has very often meant covering of the deteriorated concrete pavement with a thin asphalt layer. This measure has been regarded as temporary, but due to scanty financing it has often been the only possibility. On the other hand, also wide rehabilitation projects have been succesfully executed in the United States where the original service level of the concrete pavement has been restored. Some examples of defects and rehabilitated pavements on the American Interstate road network are shown in Figures B2-32...B2-40.

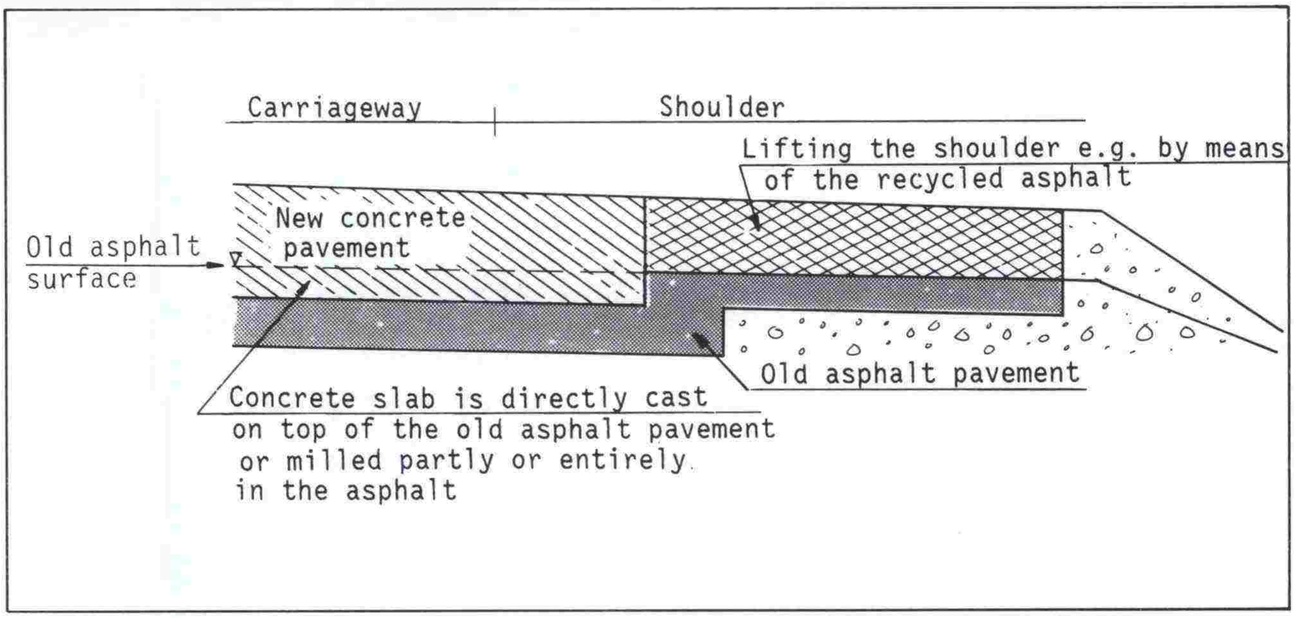


FIGURE B2-31. Principal drawing of the rehabilitation of an old asphalt road using white-topping overlay

FIGURE B2-32. An old concrete-paved urban road in Minneapolis, repaired by full depth patching (photo JR/1987)

FIGURE B2-33. A detail of a patched street pavement, Minneapolis

FIGURE B2-34. A rehabilitated concrete pavement of more than 20 years of age, State Highway 169, Minneapolis

FIGURE B2-35. A detail of a succesful patching and of a renewed joint, State Highway 169, Minneapolis

FIGURE B2-36. A worn-out, old concrete pavement, an asphalt shoulder, Thruway 87, New York

FIGURE B2-37. Asphalt patchs on a conrete pavement, Albany, New York

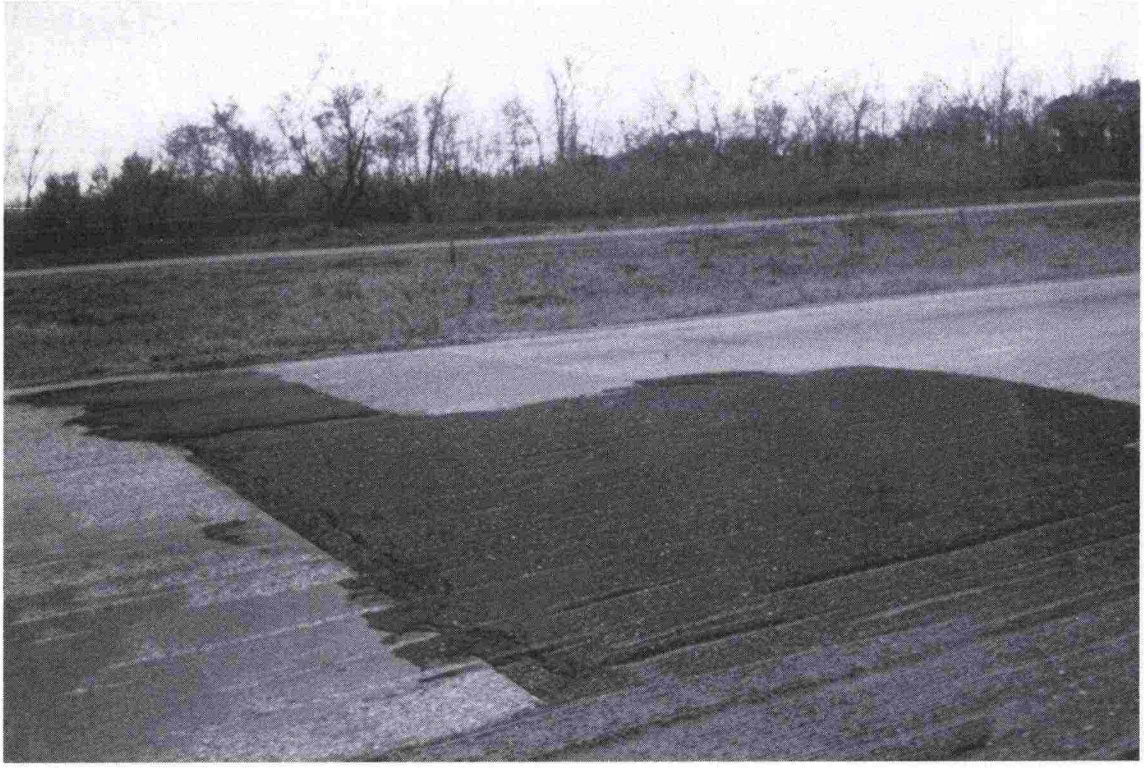


FIGURE B2-38. A bump at the culvert, a failed temporary repair, Minnesota

FIGURE B2-39. Severe D-cracking in the pavement, Minnesota

FIGURE B2-40. A repaired wild crack, Minnesota

B 282 Experiences from Central Europe /63/

Concrete pavements have a long history also in Central Europe and thus they have also experiences of damage and rehabilitation. The general impression is not, however, that of a crisis as in Northern America; the situation has been better under control. The lack of up-to-date rehabilitation instructions has, however, been evident also in Central Europe at the end of the 1970s and thus in the early years of the 1980s manuals on maintenance and instructions of design have been published in most countries where concrete pavements have been built. In connection with this explicit survey the manuals on maintenance published in England (1986,/1/), Germany (1985,/3/), Switzerland (1985, /4/) and in France (Cembureau/ Paris (1982, /2/) have been studied. The instructions have been prepared in co-operation and so they teach a similar kind of practice. The importance of constant maintenace is emphasized in all countries and the pressure to design new concrete pavements more and more durable without compromising in dimensioning or details is mutual.

Concrete pavements built on main roads in the 1930s are still trafficked in the German Democratic Republic (Figure B2-41). It has been necessary to decrease the speed level of these roads due to regular faulting of joints, otherwise the slabs are in a considerably good condition. Rehabilitation of the worst sections is undertaken as a repavement by removing the old pavement.

FIGURE B2-41. Concrete pavement built in the German Democratic Republic in the 1930s (Berlin-Dresden). Cracking and warping of slabs occur, the service level reduced

The repairing methods connected to rehabilitation are commonly used in Germany. Worn-out pavements are crushed and stabilized onto the subbase. Especially on motorways the service level is not allowed to decline considerably before repaving.

The aging concrete infrastructure (concrete roads, concrete tunnels, bridges) causes concern in Switzerland and Austria and great importance is laid on maintenance - and at least up till now with success - because the best concrete pavements of the world can be found there.

The use of concrete pavements is most common and their condition most variable in Belgium. The condition of new main road pavements is excellent, the service level of older main roads has declined and local repairs are abundantly needed. The condition of concrete pavements on lower road network is very variable and a more lax attitude than that of the neighbouring countries has been taken in the required service level.

Examples of damage types and rehabilitation methods of concrete pavements in Central-European countries are shown in Figures B2-42...B2-49.

FIGURE B2-42. An old concrete pavement on regional road N2 in Belgium. Hitting joints and cracking have been repaired by replacing slabs (photo JR/1987)

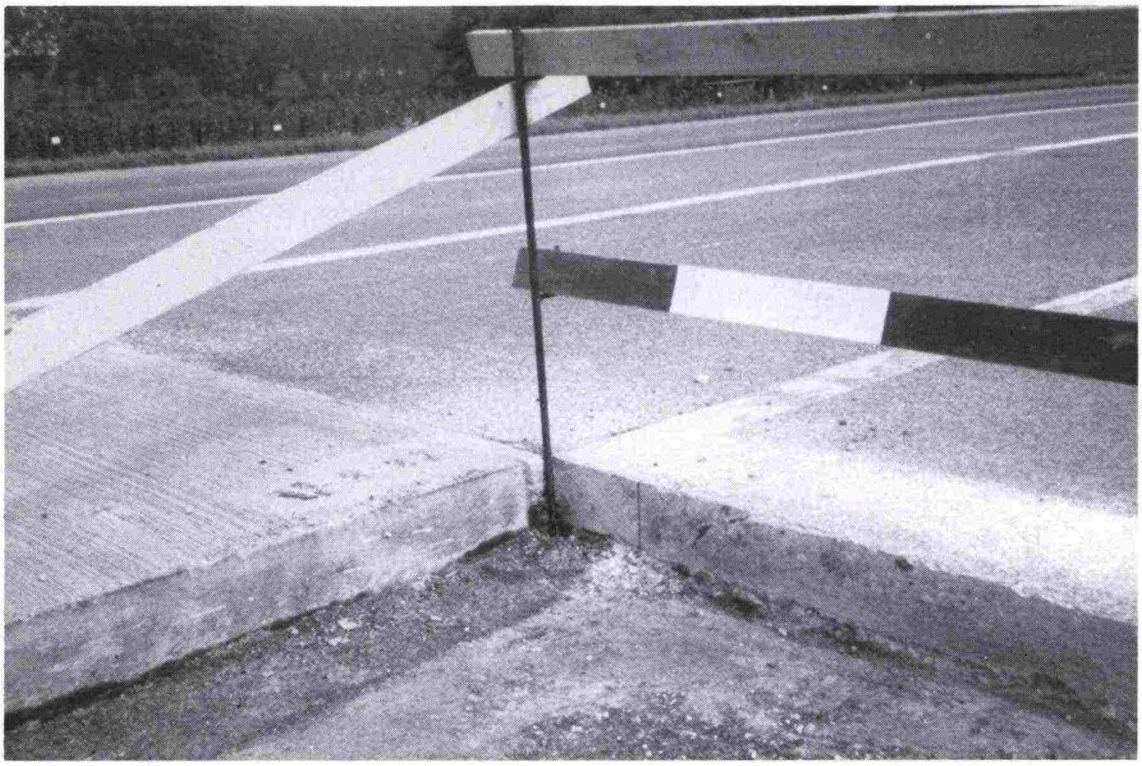


FIGURE B2-43. Full depth repair
N2 Halen-Hasselt, Belgium

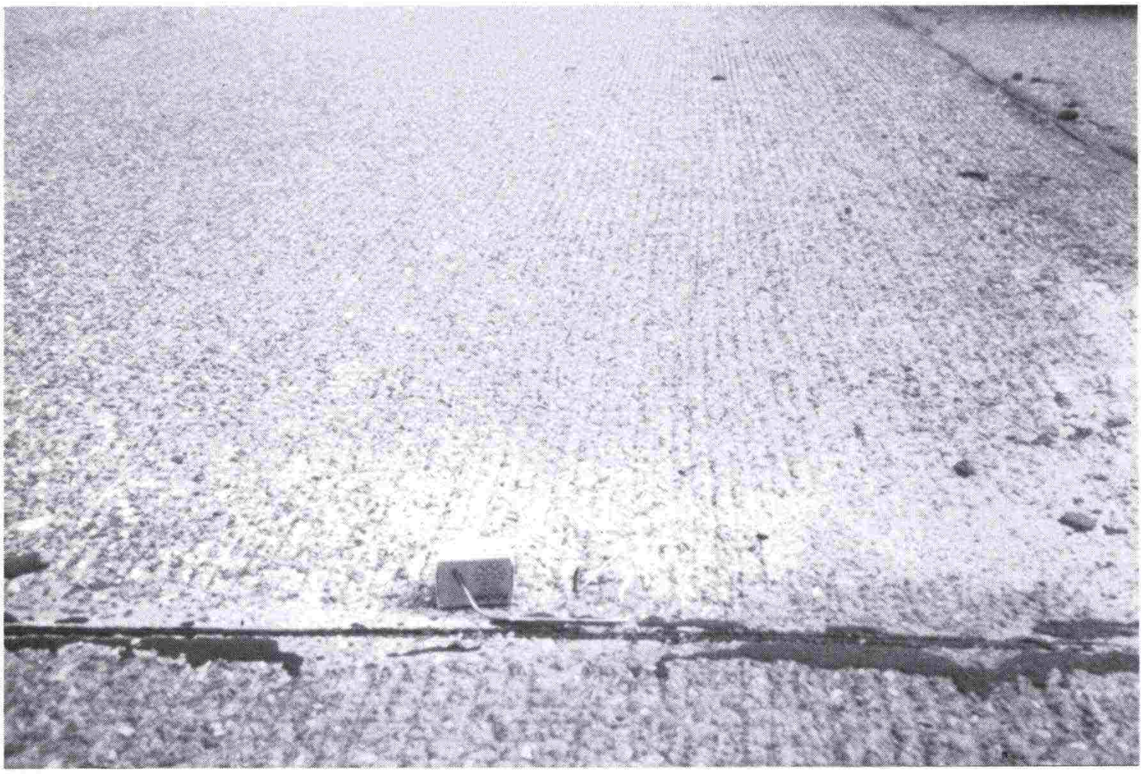


FIGURE B2-44. Pavement grooved
in connection with the repair,
the joint redamaged, N2 Halen-
Hasselt, Belgium

FIGURE B2-45. A rutted concrete pavement, ponds on the road, A1 Melk-Wien, Austria

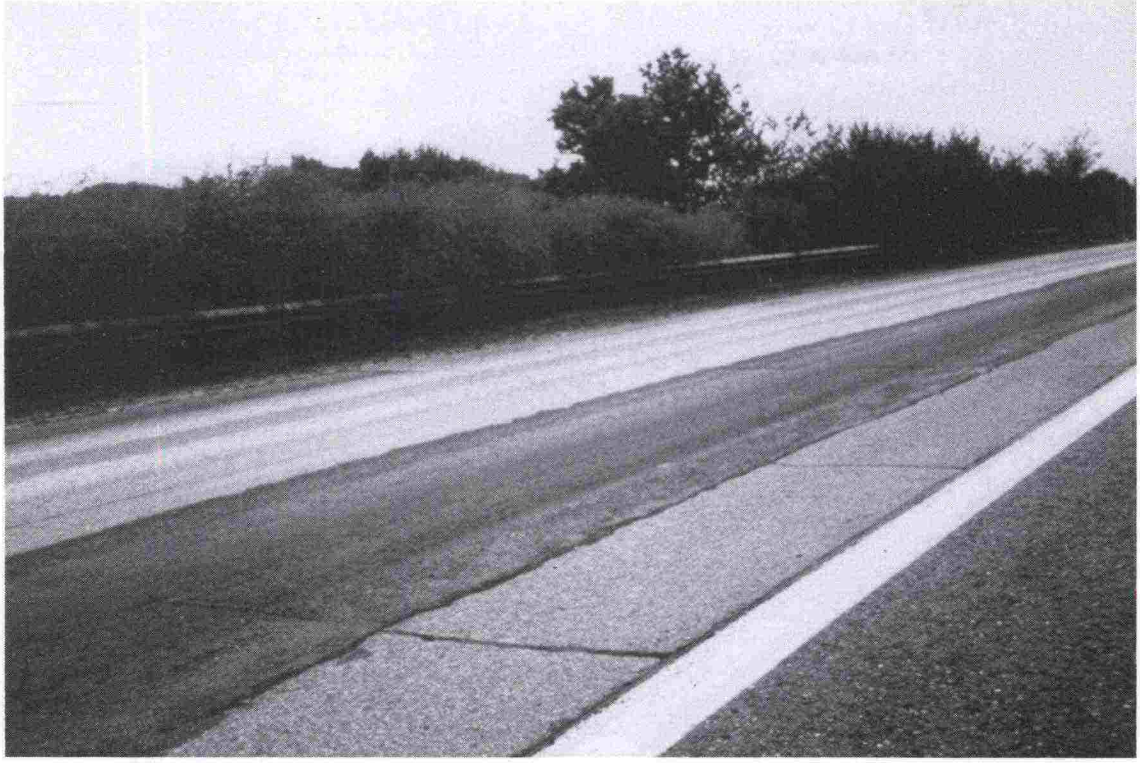


FIGURE B2-46. Gussasphalt treatment on a rutted concrete pavement, A1 Melk-Wien, Austria

FIGURE B2-47. A reinforced two-layer pavement; scaling caused by corrosion on the pavement, N13, Switzerland

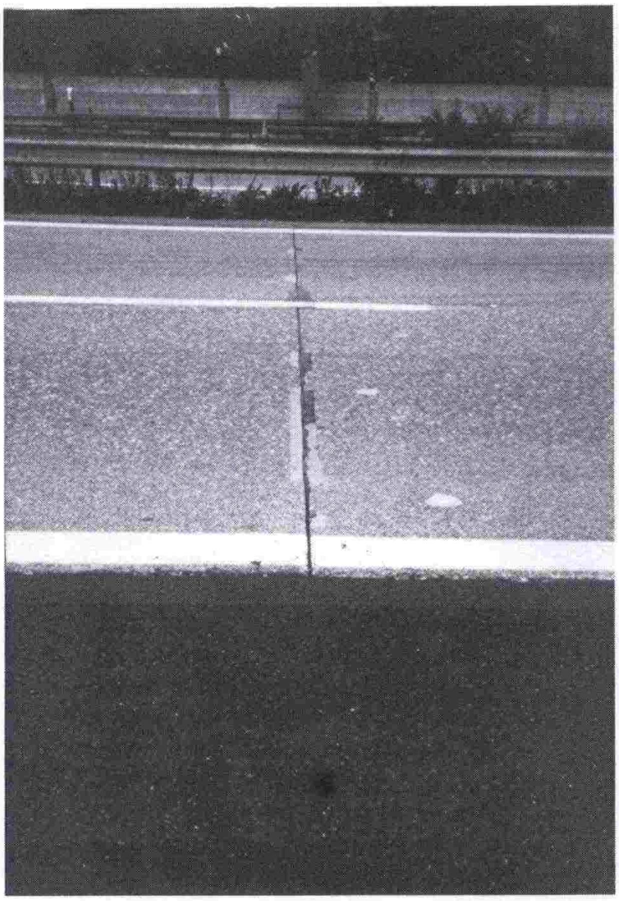


FIGURE B2-48. A damaged joint in an old concrete pavement, N1, Aarau, Switzerland

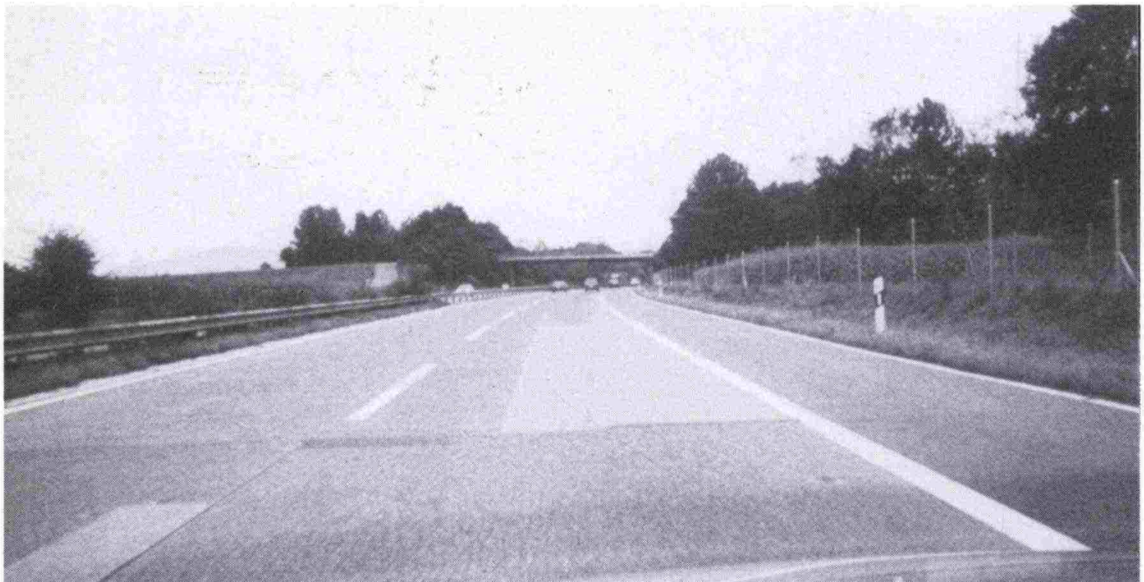


FIGURE B2-49. Concrete pavement rehabilitated by replacing slabs, N1, Aarau, Switzerland

B 283 Scandinavian experiences

There were concrete pavements in all Scandinavian countries as early as in the 1920s and 1930s. They were usually reinforced long slabs, which suffered from damage in transverse joints. With few exceptions these pavements have been paved with asphalt or rebuilt. Design principles and working techniques have been changed after the War and although the objects are few all Scandinavian countries have experiences of the durability of concrete pavements and of the need and measures of rehabilitation.

D-cracking, joint damage and different kinds of surface defects (scaling) have been common damage types of the post-war concrete pavements in Denmark. In connection with rehabilitation projects tests have been made to find suitable repairing methods. Milling and thin overlays of local damage have been of interest, but the results have not been satisfactory. Danish experiences of rehabilitation of concrete pavements have been reported by the road authorities /47/.

In Sweden rehabilitation experiences concentrate on the southern Malmöhuslän where all the oldest concrete pavements are situated. Totally more than 60 km of reinforced concrete pavements on unbound base were built in different phases between Malmö and Helsingborg in the 1960s. These sections have been paved with asphalt since 1984; the service life of the concrete pavement was about 20 years.

Before laying with asphalt lifting of slabs, milling and patching were undertaken to a considerable degree due to settlement and joint damage on these roads. The rehabilitation works were considered inconvenient although average annual costs remained rather low. A summary of rehabilitation costs published by the road authorities is shown in Figure B2-50. Concrete pavements of the 1970s as appears from the Figure have been made with unreinforced slabs on cement-treated base. An improved service level and a longer service life are expected of them. So far these sections are in a good condition (Figures B2-51, B2-52), however, the Vellinge-Malmö road built in 1972 will demand milling or other actions due to studded tyre abrasion. /64/

Reparations- och underhållskostnader av betongbeläggningarna
på vägarna E6 och E4 i Malmöhus län (1984 års prisnivå)

Väg utbyggnads-etapper	Öppn. för trafik år	Längd motorväg km	ÅDT 1984 x 1000	Reparationskostnader kkr -76 ⊗	77 F	77 S	78 F	78 S	79 F	79 S	80 F	80 S	81 F	81 S	82 F	82 S	83 F	83 S	84 F	84 S	85 F	85 S	86 F	86 S	Reparationskostn 1977-86 ΣF	ΣS	Årskostn. kr/m² F	S	ΣF+ΣS	Total uh-kostnad Kkr	Total årskostnad kr/m²	Knäckning av btg-plattor + överläggning med asfalt
				1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	23	24	25	26	23+24	1+23+24		
E6 Vellinge-Malmö	1972	11.7	18	26	29	–	–	–	48	–	–	–	433	112	421	–	–	25	114	–	–	–	125	–	1.190	137	0.63	0.07	1.327	1.353	0.52	
. Åkarp-Borgeby	1961	12.9	20	568	–	16	115	–	–	172	–	–	–	–	–	130	–	126	–	72					115	516	0.06	0.29	631	1.199	0.28	1984
. Borgeby-Lundåkra	1965	11.0	17	483	79	208	–	–	–	–	125	–	297	13	–	82	39	–	–	74	32	–	–	55	1632	432	0.33	0.25	2.064	2.547	0.69	1986
. Lundåkra-Örja	1966	7.1	16	281	5	42	–	–	–	–	–	–	48	–	–	–	238	–	327	–	294	–	–	13	912	55	0.86	0.05	967	1.248	0.59	
. Örja-Hilleshög	1966	7.0	16	317	134	–	295	–	68	–	–	–	–	–	–	–	–	–	–	67	–	4	546	14	1.043	85	0.87	0.07	1.128	1.445	0.60	
. Hilleshög-Görarp	1967	9.6	17	394	4	–	–	–	–	880	245	–	500	–	360	–	–	–	–	67	–	–	–	56	1.109	1003	0.72	0.65	2.112	2.506	0.86	1986
Görarp-Djurhagshus	1969	9.5	14	274	97	51	100	185	436	655	–	–	325	–	–	57	–	38	–	121	–	33	–	20	1.018	2.168	0.67	1.43	3.186	3.460	1.34	1986
Djurhagshus-L.[illegible]	1965	3.7	14	162	196	–	–	12	442	6	114	–	–	–	297	–	334	–	–	–	–	33	–	47	1.383	98	2.34	0.17	1.481	1.643	1.33	
E4 Vala-Hyllinge	1978	5.8	15						–	–	–	–	–	–	–	–	–	–	190	–	444	–	237	–	871	–	1.17	–	871	871	1.17	

F - Fogning och reparation av vilda sprickor.
S - Reparation av sättningar mm.
⊗ Före 1977 har inte fogningsunderhållet särredovisats.

FIGURE B2-50. Summary of the latest rehabilitation costs of Swedish concrete pavements /53/

FIGURE B2-51. Concrete pavement built on the road section Vellinge-Malmö, Main Road E 6, east of Malmö in Sweden in 1972. Ruts of about 20 mm, joints have broken at ruts here and there, condition and driving comfort good (photo JR/1988)

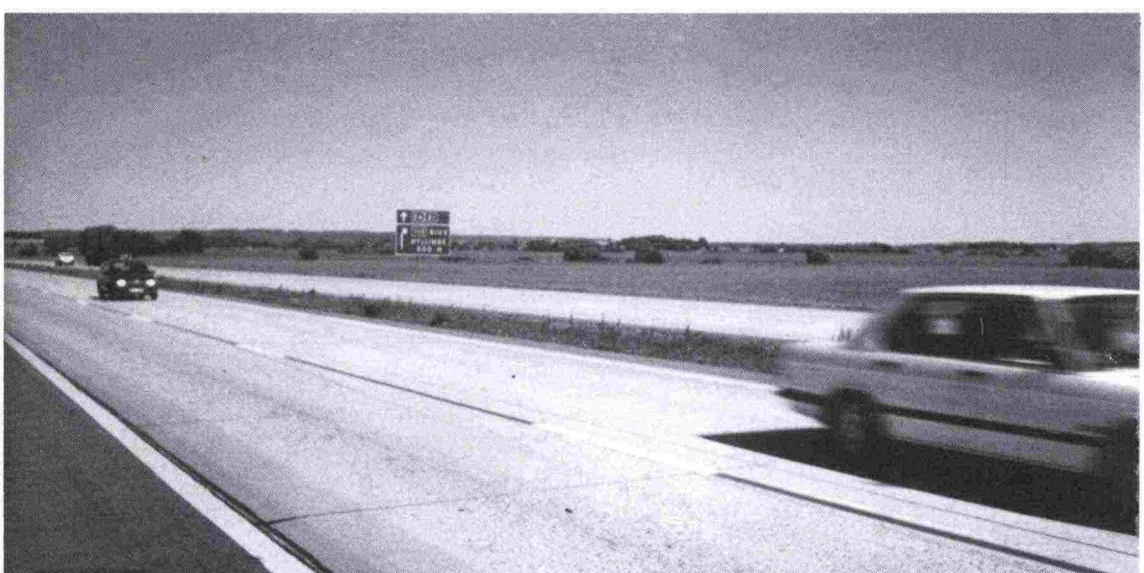


FIGURE B2-52. Concrete pavement on the road section Väla-Hyllinge, E4, Helsingborg built in 1978. Tests to move driving lines are being made in summer 1988.

Norway has long traditions of concrete pavements in bridges and tunnels; experiences on roads are concentrated on the Vestfold county, where concrete pavements have been built on the Main Road E4 both before and after the War. Rehabilitation methods are actively developed owing to the special requirements of bridges and tunnels. The latest interest has been directed to the development of wear-resistant concrete and of an applicable grinding method. Decisive progress has taken place in these questions as appears from report B3.

Finland has rehabilitation experiences of both pre-war and newer concrete pavements. Lifting of slabs and other heavier repairing measures have been undertaken on older pavements in the 1950s and 1960s. Joint repairs or small patchings with epoxy - which have sometimes also failed - have been the actions on newer pavements. Repairing or rehabilitation methods have not especially been developed nor own manuals published in Finland.

The most uniform experience of concrete pavement is available of the Ylikylä-Parainen road (see B17), which was built in 1958-59 and served for 25 years before it was paved with asphalt. It has been necessary to replace slabs with new ones or to pave them with asphalt at some culverts due to settlement or blow-ups. Cracks were formed in joints and they were patched with asphalt. No faulting of slabs occurred. Evenness was well preserved /57/. Ruts caused by studded tyre traffic and by access of a detrimental amount of water were the actual reasons for paving the road with asphalt after 25 years of service.

B 29 SUMMARY

The hard truth dawned on the concrete-using countries in the 1970s when the 'everlasting' concrete pavements built after the War began to approach the end of their service life. The causes for the fatigue and damage had to be investigated and profitable means for their repair, rehabilitation and reconstruction had to be found. In addition to the natural stress caused by climatic variations the following reasons, among others, could be stated:

- traffic stress exceeding the prognoses
- failures in design and construction
- negligence of maintenance
- inconvenience of maintenance and lack of applicable repairing methods

These observations started very lively development and investigation activities at the beginning of the 1980s. The results can already be seen:

- repairing methods have been tested and developed; new handbooks have been published
- the significance of preventive maintenance was understood
- paving techniques have been developed (e.g. breakthrough of the slip-form method)
- dimensioning of new pavements have been adjusted (thicker and shorter slabs)
- repaving methods and recycling have been developed
- cost awareness has improved, use of life-cycle costs in comparisions has become general

And the development goes on. Efficient equipment and methods for the repair of most defects and for the rehabilitation of concrete pavements can be found. The next step is to acquire the needed professional skill to be able to utilize the methods and to find proper practical applications for each country.

The special issue for Finland is the studded tyre wear of concrete pavements and the rehabilitation of rutted pavements. Among others, rehabilitation methods of ruts have been dealt with in this section of the report. Grinding seems to be the most reliable and profitable method of them, at least for the time being. Grinding of concrete pavements is a routine method in the United States, it has been applied to rut grinding, especially in Norway. In Finland ruts have been milled in connection with one project only. In the future it would be important to acquire own experiences of diamond grinding by performing wider trial works and research.

Renewing of concrete pavements has always been considered expensive and repairing inconvenient. Reconstruction and rehabilitation have their prices, the profitability of these measures depends on how wisely a maintenance strategy will be chosen and how professionally the work will be done. To a great extent, repairs will always be undertaken by hand and they will also be inconvenient due not least to the inconvenience caused for traffic. That is why the development of design and working methods are important. The need for repairs of newly designed concrete pavements will essentially be smaller and their service life longer than those of the previous generation where long slabs and the low strength of concrete have proved to be the weak points.

CHAPTER B2 REPAIR AND REHABILITATION OF CONCRETE PAVEMENTS

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CHAPTER B3
CONCRETE PAVEMENT WEAR DUE TO STUDDED TYRES

CHAPTER B 3

CONCRETE PAVEMENT WEAR DUE TO STUDDED TYRES

CONTENTS

B 3
CONCRETE PAVEMENT WEAR DUE TO STUDDED TYRES

B 30 INTRODUCTION

One of the most important qualifications of a road pavement in Finland is its wear resistance under studded tyre traffic. Concrete pavements are known to resist wear better than asphalt pavements. Wear resistance values that are 2,5...5,0 times higher than those of asphalt have been measured for usual road concretes. This fact cannot but arouse interest in finding new ways to manage the ever-worsening wear problems of pavements. But there are still many questions to be answered as far as the use of concrete in studded tyre traffic is concerned: how wear resistance of the concrete pavement can further be improved; how a rutted concrete pavement can be repaired and at what price; what kind of a wear relation compared with asphalt pavements can reliably be reached; is the wear an additional threat or a possibility for the growing use of concrete pavements.

Up-to-date knowledge from abroad is available only from Norway and Austria. Wear qualifications of concrete pavements were energetically studied elsewhere in the 1960s but after the prohibition of studded tyres- or after their decreased usage- in the beginning of the 1970s the interest in further development of wear-resistant road concrete ceased. In Finland concrete-paved test roads have been built at intervals of 2-5 years and the qualification concepts of wear-resistant road concrete has become established at the same time, although there are still many questions to be answered. The development of wear qualifications of concrete pavements was given a new push again, when a research project was started in Norway in 1984. A mutual investigation was carried out in 1987 to adjust the Norwegian and Finnish experiences for further development of the road concrete; the Norwegian and Finnish cross-studies of equal concrete mixes were performed by test track runs in both countries.

Wear resistance of concrete pavements primarily based on the Norwegian and the latest Finnish data is dealt with in this section of the report; a general view on the wear resistance criteria of pavements under studded tyre traffic is also given.

B 31 PAVEMENT WEAR AND STUDDED TYRES

B 311 Wear develops into a problem

As a matter of fact the wear of road pavements was an unknown conception all over the world till 1960. The wear was considered polishing and weakening of friction qualities of old pavements. But the use of steel studs spread like wildfire in all cold countries since the beginning of the 1960s and this soon created a new kind of a problem - rutting of pavements. Studs cut down the braking distance on icy roads and improved clearly the sense of safety in drivers, although their total effect on the traffic safety was and still is critically discussed. When studded tyres grew in use due to lack of restricting rules, the disadvantages became quickly evident. Road markings wore off and ruts began to form on tyre lines. Ruts reduced driving comfort, decreased operating speed, caused splashing of water and danger of aquaplaning and reduced safety especially in lane-changing situations. In the 1960s the wear problem was first intensified in countries where traffic-flows were the highest as in North America and in Central Europe. This was altogether a new kind of problem, there were no conceptions nor means to study the rutting, there were no means to repair it. A decade of lively research activities followed. Studded tyre wear was investigated especially in road laboratories in Minnesota and Ontario, in Europe the problem was studied in Germany, Switzerland and Austria. Also Scandinavia had an energetic part in the investigations. A wide wear investigation programme both on test roads and test tracks was executed in Sweden as early as in 1963 - 64. A test track was built in the road and traffic laboratory of VTT also in Finland as early as in 1966.

The intensive investigation stage clarified the conditions of good wear resistance of both concrete and asphalt pavements. But at the same time it became evident that studded tyre traffic is a severe threat for the service life of pavements; repaving and rehabilitation costs are sharply increasing. This entailed in a stern restriction of the structure and use of studs. The use of studs were totally prohibited in many countries at the beginning of the 1970s.

B 312 The use of studs is restricted

To control the wear of pavements restrictions of the size and number of studs, of their installation in tyres, of their operating time etc. were stipulated all over the world as early as in the mid-60s. However, these instructions gave only breathing-space, because increasing traffic made the situation worse again and stricter stud instructions had to be made. By 1975 most stud-using countries had either voluntarily or with stipulations banned the use of studs.

A total prohibition of studs is in force only in ten states in the United States - only five of them are northern states. The decline in studded tyre use has mainly been achieved voluntarily, there is no winter tyre compulsion, belted tyres facilitate winter driving and the policy of 'bare roads' is adapted; also the public opinion is against studded tyres. The studded tyre wear of pavements is no more a significant problem in the United States.

The use of studded tyres was prohibited in Canada as early as in 1971 after thorough investigations and public discussions. A complete prohibition came into force in Germany in 1975. There have been pressures to allow the use of studs but they have been prevented by new investigations, last in 1985/40/. The use of studs is prohibited on the main road network in Switzerland, but allowed on inferior roads. It is understandable that the utilization rate remains low in such a situation. The use of studs is allowed on all roads in Austria. The utilization rate has, however, decreased -only to about 10 % of cars - and the studded tyre wear is no more an acute problem. The use of studs has not been significant in the Soviet Union or in other Eastern European countries.

Scandinavia will soon be left alone to struggle against rutting. The use of studs is allowed and the utlization rate is high. In Norway nearly all passenger cars and about 60 % of heavy vehicles use studs from the 1st of October to the 1st of May. However, the authorities try to shorten the utilization time (it ended on April 10 in 1988). In Sweden the use of studs varies from about 50 % in the south to 100 % in the north. Heavy vehicles have not been equipped with studs; the use was completely prohibited in 1989. The utilization time in Sweden is from the 1st of November to the Easter. In Finland almost all passenger cars and over half of the heavy traffic use studs. A compulsory use of winter tyres is in force in Finland and Norway, no corresponding compulsion has been stipulated in Sweden.

The studded tyre wear of pavements seems to be a permanent factor in road management in Sandinavia - especially in Norway and Finland, even to the extent that the main attention is paid to wear in the road technique and policy at least as long as a better balance between wear and wear resistance will be reached.

B 313 Regulations for the use of studs in Finland

It has been legislated that the the point of a steel stud must not be sharp or tubular (Figure B3-1). The stud protrusion from the tyre surface must range between 0,5 and 1,5 mm in passenger cars and between 1,0 and 2,0 mm in trucks. The static stud force must not exceed 0,12 kN measured by a protrusion of 1,2 mm in a passenger car tyre and correspondingly 0,35 kN measured by a protrusion of 1,7 mm in a truck tyre. There must be 10...22 studs pro each 300 mm of the tyre surface. In practice 90...120 studs divided into 6 rows on the tyre surface edges are installed in passenger car tyres (Figure B3-2). The studs must be installed on all tyres, the outer double tyres of trucks can be stud-free. Studded tyres can be used from the 1st of October to the 30th of April, two weeks shorter times are recommended in Southern Finland. The actual winter tyre compulsion concerns only December, January and February.

The above stipulations concern only so called fixed studs. The intention of the stipulations is to support the security aspect of studs but also to avoid disadvantages so that "they do not essentially damage the road surface". The wear is considerably affected especially by adjusting protrusion and stud force, but their reduction decreases also the grip of fixed studs. The development of the sleeve stud or of the security stud into a market product may open new possibilites to decrease the wearing stress without compromising with security angles.

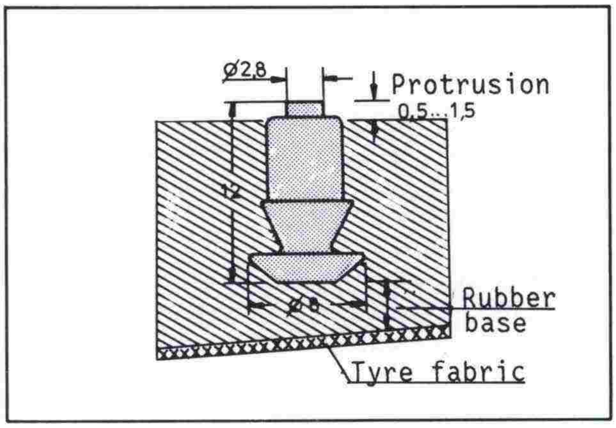


FIGURE B3-1. Structure of a stud

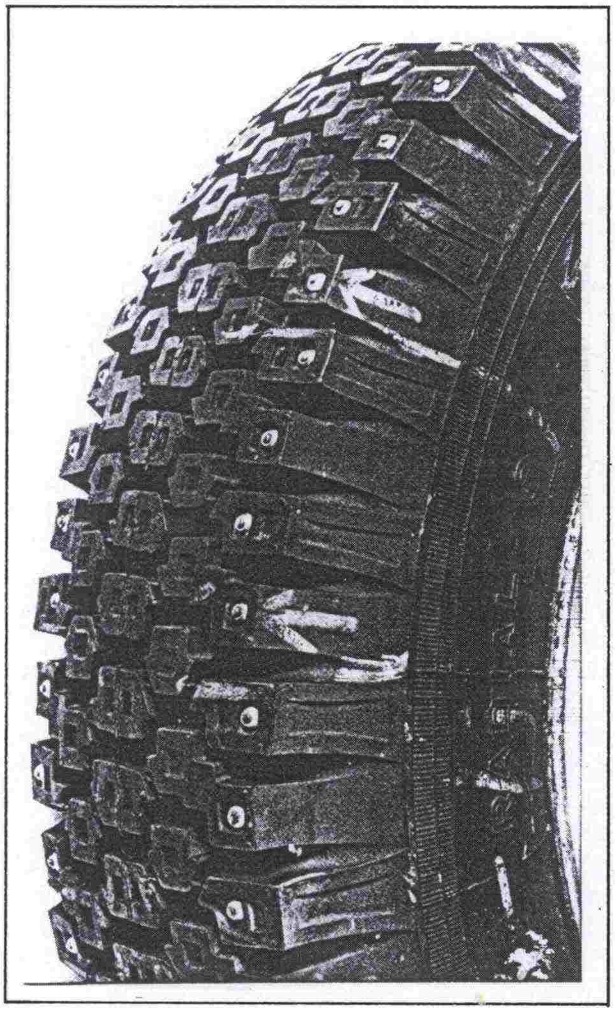


FIGURE B3-2. Studded winter tyre

B 32 PRINCIPLES OF PAVEMENT WEAR

B 321 Emergence of wearing stress

The wear effect of studs can be explained as follows:

- when touching the road surface the stud hits stone grains and breaks them
- when pressed against the road surface the stud skids sideways and rubs the pavements with the stud force
- when loosening from the pavement the stud scratches the stone or binder mastic

In normal road traffic all these components are included and the wear occurs as a combined effect. The impact of the hit will be strengthened by an increase in driving speed, the rubbing will be strengthened in curves and the scratching for example in situations of braking and acceleration. On circular test tracks the driving is merely curve driving and thus it is mostly rubbing that causes the wear effect. It depends on the surface qualities of the pavement how the hit, rubbing or scratching will be able to penetrate into the pavement.

The pavement-wearing stress can be considered to be the total product of the stud force and of the amount of stud stabs according to Figure B3-3. In practice both components are influenced by so many factors that establishing the wearing stress in advance by mathematical calculations gives inaccurate results. However, an investigation of the different factors contributes to understand why the wear is so variable in practice. The stud force is influenced at least by the following factors according to Figure B3-3:

- the protrusion of the stud from the tyre

The greater the protrusion the greater the stud force. In a worn tyre the protrusion will increase.

- the stud quality

The stud size and especially the amount and size of the attachement flanges increase the stud force.

- the tyre quality

The harder the rubber and the thicker the rubber layer under the stud head the greater the stud force.

- the magnitude of the wheel load

An increase in the wheel load results in an increase also in the stud force: compared with a passenger car stud a 10-15-fold stud force of a truck stud has been established in some investigations.

- driving speed

An increase in the driving speed results in an increase in the wear effect in proportion to the square of speed. Driving speed has a decisive effect on the extent of the wearing stress.

- temperature

Temperature has a diverse effect on tyres and air pressures. It is a combined effect that stud forces increase with a decrease in temperature.

The amount of stud stabs depends on traffic and tyre factors and on the studding procedure. At least the following influental factors can be seen:

- the amount of studs per tyre and the tyre size

These factors determine how many times the stud hits the road surface when the vehicle is moving. Generally, a normally studded passenger car tyre is calculated to hit the road 65 times/m; the corresponding figure of a truck tyre is 30 times/m.

- the amount of studded tyres per vehicle

All passenger car tyres are studded but the use of studs in trucks is less common (approximately 3,5 tyres/truck)

- the amount and composition of traffic

The total traffic flow during a studded tyre season and the share of heavy vehicles are naturally the most important variables in defining the wearing stud stress. The wearing stress of heavy vehicles is 2,5 - 4,0 times higher than that of passenger cars.

- the utilization rate and time of studs

The utilization rate of studs in cars is nearly 100 % in the middle of winter, but the rate remains clearly lower in autumn and in spring. The utilization rate of studs in trucks is about 80 % in northern Finland and about 40 % in southern Finland. Studs are used for more than half a year, 6-7 months annually.

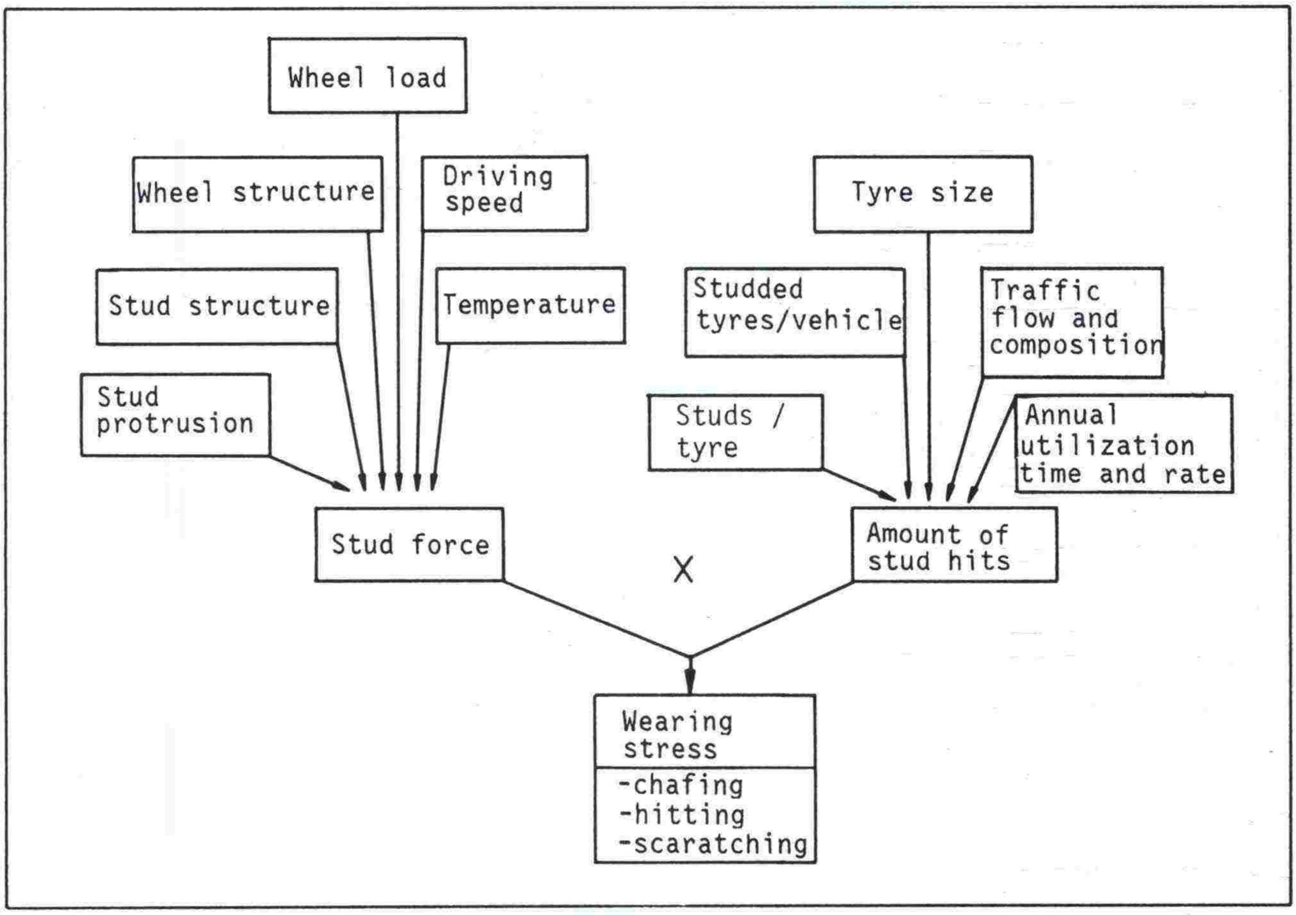


FIGURE B3-3. The wearing stress of the stud and causes affecting the wear

B 322 Wear resistance

The fact how well the pavement endures the above described wear stress of studded tyre traffic depends most essentially on the inner structure of the pavement but also on the traffic environment and climate (Figure B3-4).

General characteristics of the wear-resistant pavement are:

- a rigid and homogeneous aggregate

Wear of a pavement is mainly wear of aggregate. That is why the aggregate quality has a decisive effect. The wear-resistance of an aggregate is best described by the improved impact value and the abrasion value, also the LosA-value is suitable for a rough estimation. All these values are as small as possible in a good aggregate.

If the aggregate contains different kinds of aggregate grains as to their wear resistance the pavement wears unevenly. Thus the homogeneousness - along with the wear resistance - of an aggregate is most important for a wear-resistant pavement.

- a great share of rough aggregate on a pavement surface

The wear resistance of stone is generally always greater than that of a binder and of a mastic composed of fine aggregates. In practice it has been proved that addition of rough aggregates can improve the wear resistance of the wearing surface. This means a gap in the grading curve of the aggregate, or texturing the pavement with a surface treatment with chippings. As far as concrete is concerned washing the fresh pavement surface reveals the rough aggregates and reduces the initial wear.

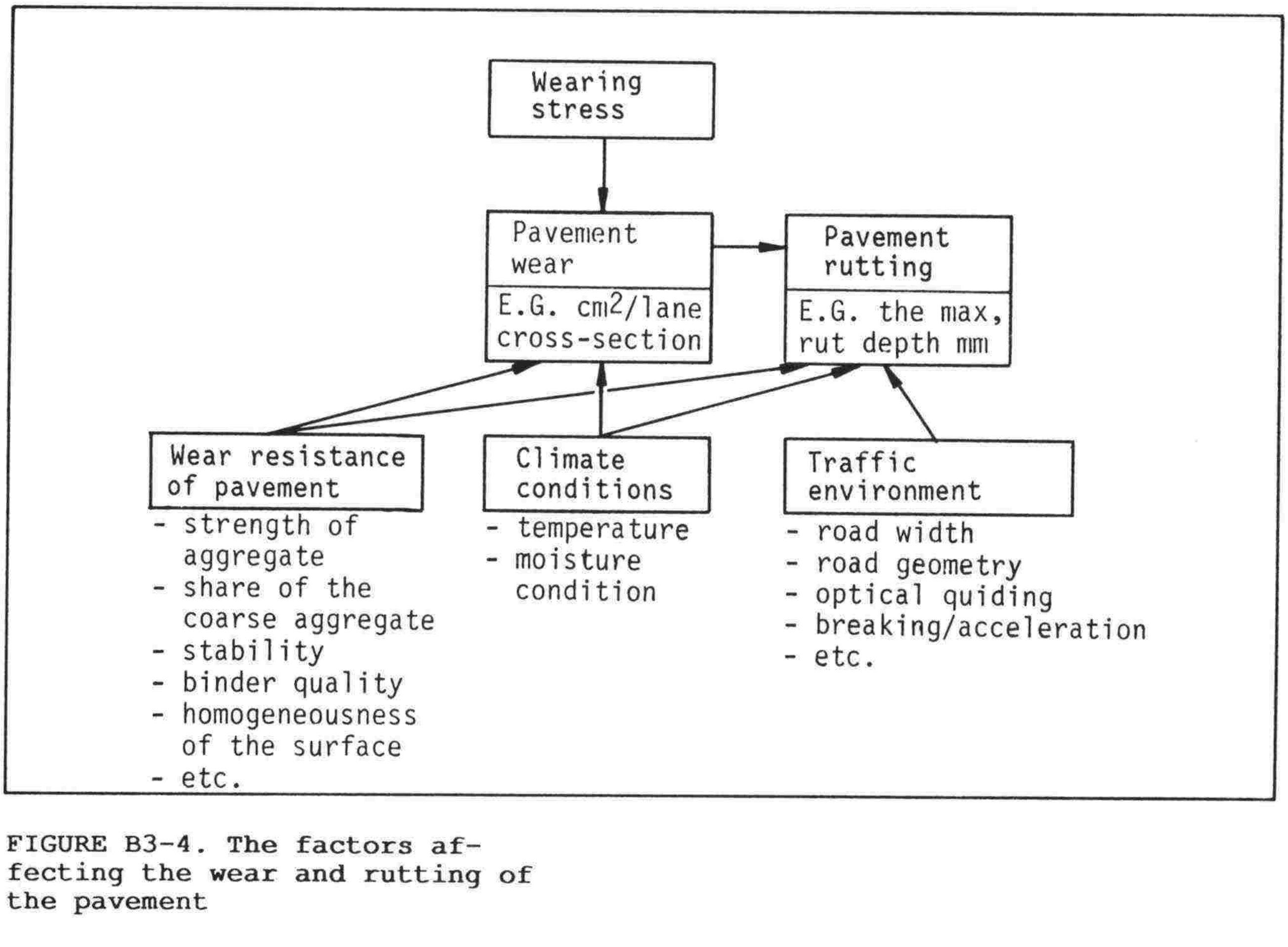


FIGURE B3-4. The factors affecting the wear and rutting of the pavement

- a good stability of the pavement mix and a firm bond between the aggregate and the binder

As far as the wear is concerned the task of both fillers and additives is to form a firm "bed" for the wearing aggregate so that the aggregate grains remain reliably attached to the bed in all conditions. The better wear resistance of the concrete pavement is partly due to the fact that the cement bond is rigid and the wear is "pure" wear. Asphalt pavements have a flexible bond; hence loosening of aggregates and deformation of the pavement mix occur as well.

The wear qualifications of both pavement types can greatly be influenced by changing the composition of the mix as is shown later in connection with concrete pavements.

The climate affects the wear of the pavement through moisture and temperature (Figure B3-4). A damp pavement wears off more quickly than a dry pavement. This concerns both pavement types,(Figure B3-5). The annual wear is thus greatly influenced by the fact how many days there are during the winter when the road surface is bare and the pavement wet. The winter temperatures do not affect the wear of concrete pavements, instead, the wear of asphalt pavements is at its lowest in temperatures of -5...+5°C and is multiplied in severe frost (Figure B3-5).

B 323 Wear - rutting

Rutting of the pavement means that pavement material is loosening from the road surface. The amount of material ground away per one road kilometer per vehicle is regarded as a measuring unit of the wear. Internationally the figure is known as a SPS-value:

1 SPS = 1 g/km/vehicle (or = 1t/km/10^6 vehicle) = the specific wear of the pavement

The specific wear can be investigated in the field by measuring areas of a worn cross-section and traffic flows (Figure B3-6). SPS-values of different pavement materials are defined by accelerated wear tests in laboratories. Typical SPS-values for standard aspahlt or concrete pavements could be:

SPS=30±5 g/km/vehicle asphalt
SPS=10±5 -"- concrete

The actual inconvenience is caused by concentration of the wear on wheel paths, i.e. by the rutting of the pavement. The rut depth is a measuring unit of the rutting phenomenon (Figure B3-6). The rut depth formed during one year in relation to the cross-section traffic is generally used as a rating index illustrating pavement qualifications.

Uo [mm/year/1000 ADT] = specific rutting. The rut depth formed during one year in relation to excesses of studded tyres are also used as a rating index: Uo [mm/year/10^6 studded tyre veh.] = specific rutting.

When using these rating indices it should be noted that rut depth usually refers to the average rut depth of the road section concerned or that the term, 1000 ADT, does not directly describe the amount of the studded tyre traffic stressing the lane. However, it is suitable for the proportioning of the measured rut depths to the traffic because the ADT-value is always available. On roads with one carriageway the ADT-value is used as such, as an amount of two-way traffic; coefficients based on the lane number are used on motorways. As rating indices describing the magnitude can be mentioned:

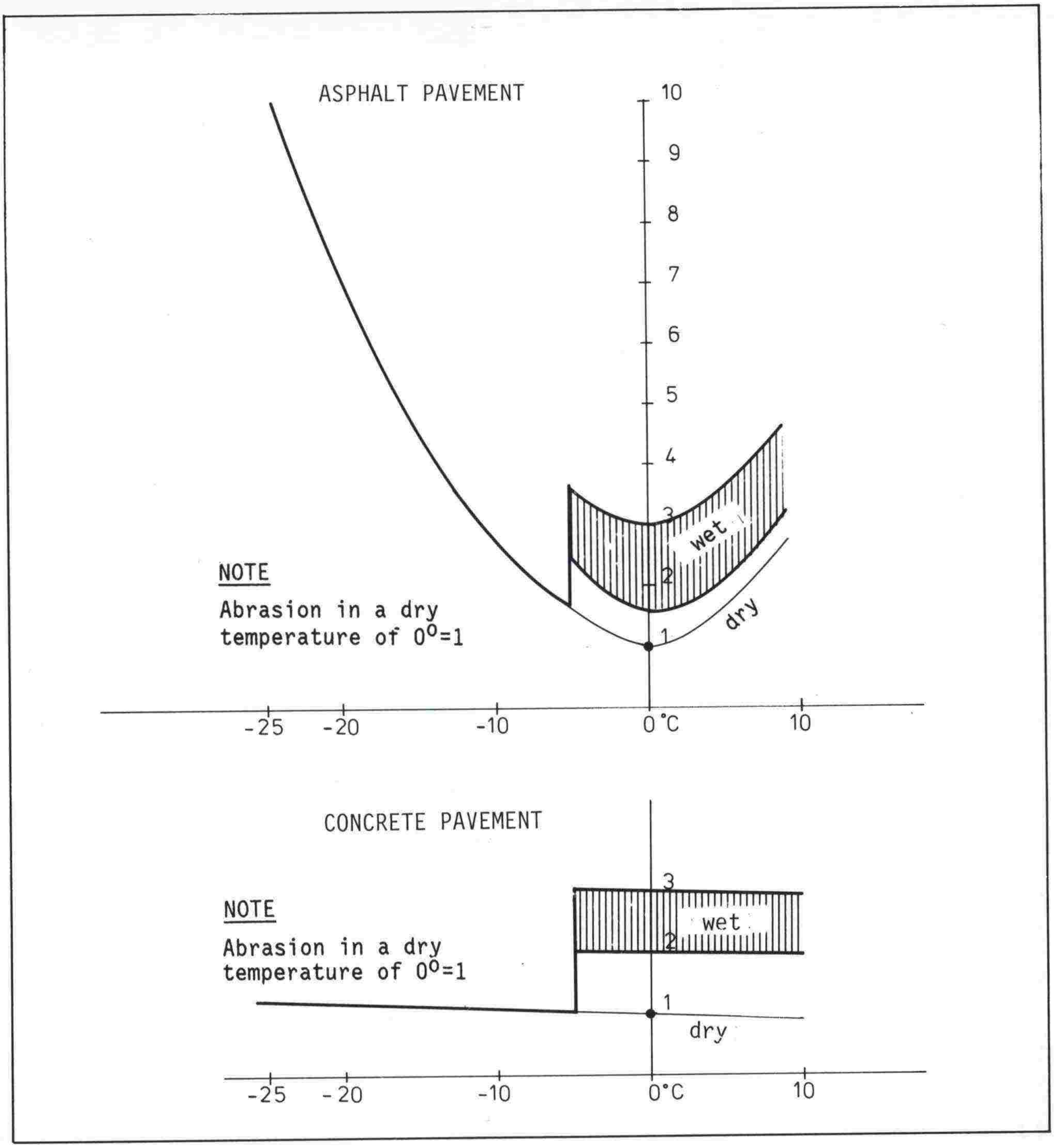


FIGURE B3-5. The relative wear of pavements when dry and wet in different temperatures /29, 25, 39/

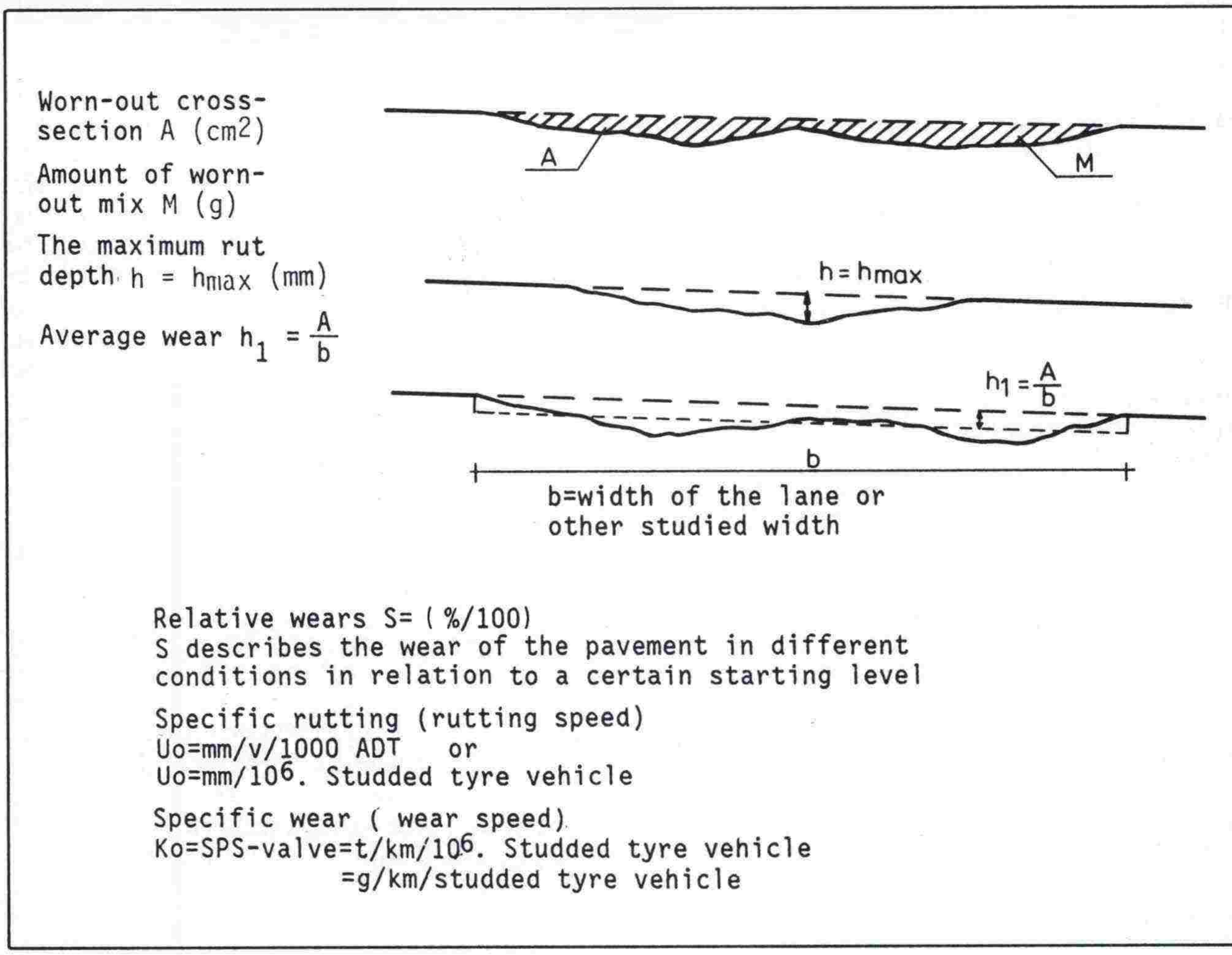


FIGURE B3-6. Concepts regarding wear and rutting

asphalt:
addition of a rut 0,45 ± 0,15 mm/year/1000 ADT

concrete:
addition of a rut 0,12 ± 0,05 mm/year/1000 ADT

Especially the road environment where the pavement is situated affects the rutting of the pavement according to Figure B3-4. The same pavement on the same road ruts in a different way on different parts of the road depending on e.g. the following facts: the width of the lane, the slack margin for the car, the width of shoulders, the line and grade of the road and the optical guidance. These factors either produce identical driving lines or disperse them at the total width of the lane. Development of the rut depth can thus be retarded by wider roads, a better road geometry and by other corresponding measures. Also many psychological reasons affect the rutting, for example it has been proved that a visible rut concentrates driving lines just into the rut which again accelerates rutting.

Rutting on concrete pavements is entirely caused by wear, whereas on asphalt pavements it is regarded that approximately 20 % of the rut depth is caused by deformation of the pavement and structure during the summer. Abroad where studs are not used at all the most heavily-trafficked asphalt pavements rut detrimentally merely because of thermal deformation. Deformation has spoiled a new asphalt pavement in single cases also in Finland. Elimination of the deformation danger is one of the reasons when a change of the most heavily-trafficked asphalt pavements into a concrete pavement is considered in many countries.

B 324 Allowed rut depths

Definiton of the allowed maximum values for rut depths is a question subject to consideration. Generally, gathering of water in the rut is regarded as a primary viewpoint. Although a 5-mm-rut is often considered detrimental due to splashing of water, only ruts of approximately 20 mm are generally regarded as dangerous due to acquaplaning.

The crossfall of the road contributes greatly to the fact how quickly an injurious amount of water is gathered in a rut and how deep a rut can be allowed. Due to insufficient financing it is necessart to make also road politics on the allowed rut depths; the tolerance will be set on the level of financing possibilities.

A general principle of the Finnish pavement design of public roads /27/ is that the average depth of the outermost rut before repaving measures should be at least equal to what is shown in Figure B3-7:

A 13-32 mm deep rut would thus be a threshold to repaving actions depending on the traffic flow and on the speed limit of the road. Excesses of limit values on the paved road network are few (about 1,5 %) after the paving season in autumn, but by the spring the actual rut depths on pavements to be repaired are considerably greater than the allowed values. In Finland about 700 km of roads are repaved every year merely because of rutting which costs more than 100 million marks.

B 325 Investigation of wear qualifications

The most reliable picture of the wear resistance of pavements and of rutting is obtained by building test pavements of real materials in a real scale in real traffic conditions and by studying wear and rutting by means of field measurements. Pavements can also be developed on the basis of the rut measuring data available from the paved road network. This is also possible as far as asphalt pavements are concerned in Finland, but concrete pavements are so few and their annual wear rate so small that accelerated laboratory tests are necessary.

Rut depth (mm)

ADT \ speed limit	<60	80	100	120
>1500	32	23	20	16
1500-6000	29	21	18	14
>6000	25	18	15	13

The average rut depht of a road section measured with a rut indicator and added by the average wear of the time between the measuring date and the paving date should be at least the valve shown in the table when decisions about the renewal of the pavement are to made.

FIGURE B3-7. Criteria for repaving a rutted pavement according to the publication 'Päällystesuunnittelu 1984' /27/

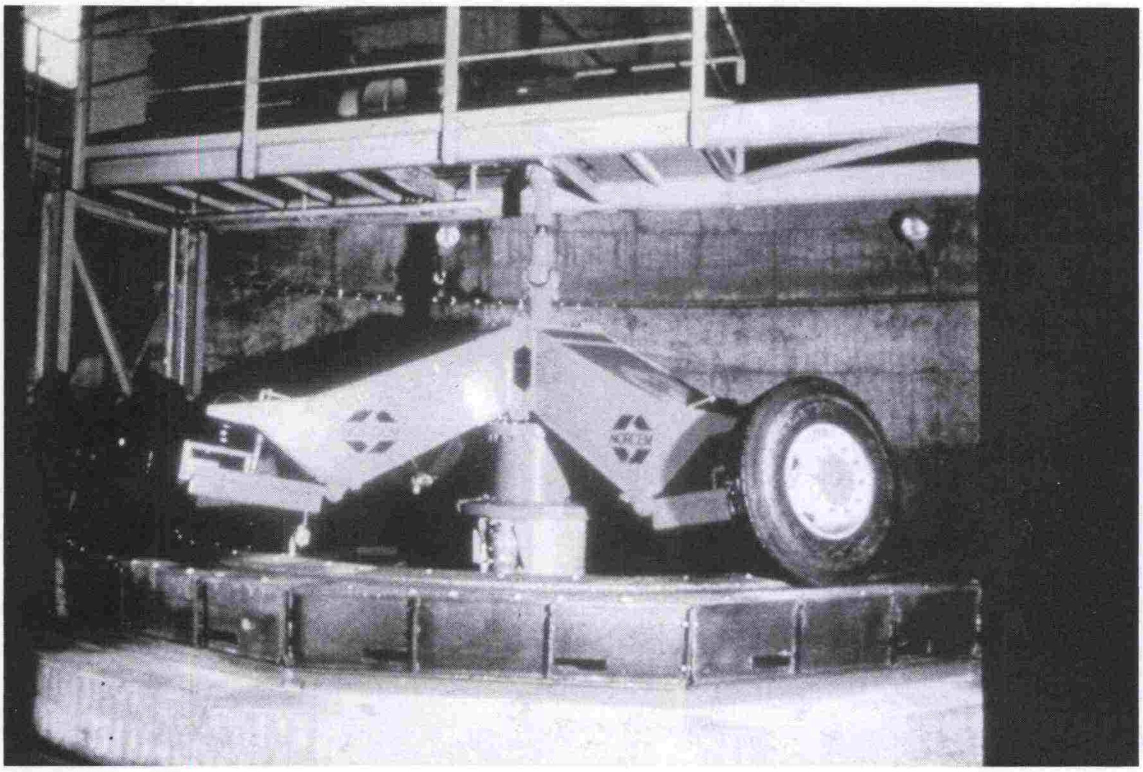


FIGURE B3-8. Wear test track in the laboratory of Norcem in Slemmestad, Norway /15/

Studded tyre wear has been investigated in laboratories since the 1950s. A typical test device is a horizontal wear test track (Figure B3-8) where a machinery rotates studded tyres along a cycle of a few meters where the test pavements have been placed as segments (Figure B3-9). In principle equal devices were taken into use in tens of laboratories all over the world when the use of studs became general. The test tracks in Finland and Norway may be the only ones in active use at the moment. Because results of test track runs made on Norwegian and Finnish test tracks are studied later on, the characteristic information on these two is given in Figure B3-10. The Norcem track is heavier and it is especially planned for wearing of concrete slabs. The test track of VTT has been planned for asphalt pavements; the small depth of the form (6 cm) has hindered research of concrete pavements. The thickness of the slab could be raised to 9 cm in the latest comparision investigations.

FIGURE B3-9. A wearing conical tyre in the test track of Norcem /15/

	THE FINNISH TEST TRACK	THE NORWEGIAN TEST TRACK
Taken into use	1966	1985
Location	VTT, Road and traffic lab., Espoo	Norcem AG, Road lab. Slemmestad, Norway
Track, diameter	3,66 / 3,06	6,0 m
speed	32 km/h	56...70 km/h
wheels	4 pcs	4 pcs
wheel load	3,3 kN	25 kN
Tyre, type	Normal ha-tyre NR 09 165 x 13	Vulganized, conical truck-tyre
studs	120 pcs	400 pcs (12/17 mm)
pressure	200 KPa	750 KPa
Track form, number	6 pcs	12 pcs
depth	6 (9) cm	25 cm
Driving conditions, moisture	Wet / dry	Wet / dry
temperature	n. +5^{o}C	No cooling
Range of use	Asph. (Conc.)	Conc.

FIGURE B3-10. The Finnish and Norwegian wear test tracks (horizontal tracks)

Also other kinds of wear test tracks are used. The best known in Finland may be the vertical track of Neste Oy which was at least once used for research of concrete samples /25/. Also so called bearing tracks - which are horizontal tracks - have been used for research of wear qualifications. The laboratories of Neste Oy in Porvoo (diam. 3,7 m) and of LCPC in France (diam. about 30 m) have this kind of a device. Mainly a milling device measuring wear qualifications of concrete for research of arctical applications has been taken into use in the concrete and silicate laboratory of VTT in 1987. The Tröger-device developed in Germany is perhaps the best known laboratory device where a test sample rotating horizontally around its axle is shot at with hard metal pins. The device imitates considerably well the wear caused by studded tyres. The same concerns also the wear device of drilling samples ('Sisto'-device) developed by the road and traffic laboratory of VTT where small wheels equipped with standard studs wear the drilling sample from the side. Neither of the available testing devices describes satisfactorily the actual road wear. Hence the results of test track runs are to be calibrated to field measurements before making any final decisions. However, maximum ruts on a pavement can be worn on test tracks in 1 - 3 weeks and thus essentially accelerate the investigation and development of more durable pavement types.

B 33 WEAR RESISTANCE OF CONCRETE PAVEMENTS

Before the comparision of wear resistance of concrete pavements with that of asphalt pavements in Chapter B 34, there is reason to make "concrete to compete with itself", in other words to study those factors that make the concrete pavement wear-resistant.

B 331 The characteristics of the wear-resistant concrete pavement on the basis of earlier investigations

The concrete pavement is dimensioned to those repeated stresses which are caused by traffic loads and variations in the climate. Concrete is proportioned to a certain compressive and flexural strength without forgetting frost resistance. The construction work aims at good evenness and the surface texturing at good friction qualities.

Studded tyre wear is a strange factor in this picture. It reduces the effective thickness of the slab, demands entirely new qualities of concrete and nullifies the significance of texturing. Good wear resistance of the concrete mix is thus an essential question except for the serviceability also for the durability of concrete pavements.

Before introduction of studded tyres in the 1950s the pavement concrete mix was made of Portland cement and of local pure aggregates. The grading curve was constant and the maximum grain size great (> 40 mm). The rough aggregate had to be, at least partly, crushed. Cement was used 250 - 300 kg/m^3 and the aim was a compressive strength of about K30 at the age of 28 days. The water/cement ratio was high (> 0,50), the casting was undertaken with fixed-form machines and vibrating beams.

The frost resistance was ensured with advance tests by adjusting the amount of cement.

Although these proportionings of concrete mix did not pay attention to wear demands at all, they generally endured studded tyre wear clearly better than asphalt pavements, depending, however, on the wear qualities of the basic aggregate. Wear results of a test track run made with traditional pavement mixes in Minnesota are shown in Figure B3-11. Inspite of the relatively good wear resistance it was generally regarded that also concrete pavements wear too much, especially when repair methods were not known. When a research programme to improve the wear resistance of concrete pavements was started at the same time in many countries in the 1960s /1,2,3,5,6,41/ it was soon discovered that the quality and amount of rough aggregates (as hard as possible, crushed) were the main factors in the improvement of wear resistance. It was discovered e.g. in Minnesota that by changing the rough aggregate and with addition of cement by 15 % a 10 % better durability was achieved than that in Figure B3-11. This procedure, however, resulted in an additional cost of 25 % compared with traditional concrete mixes. A Swedish investigation proved that a decrease

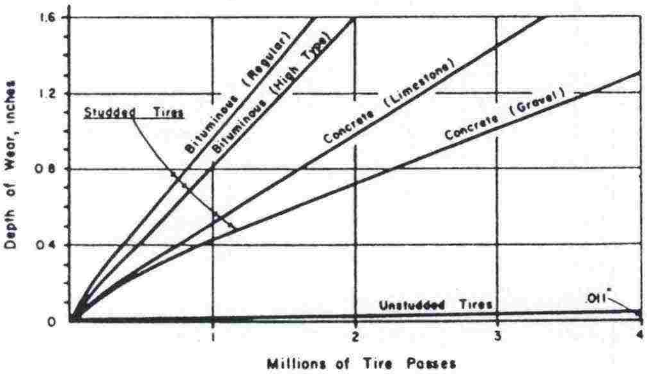
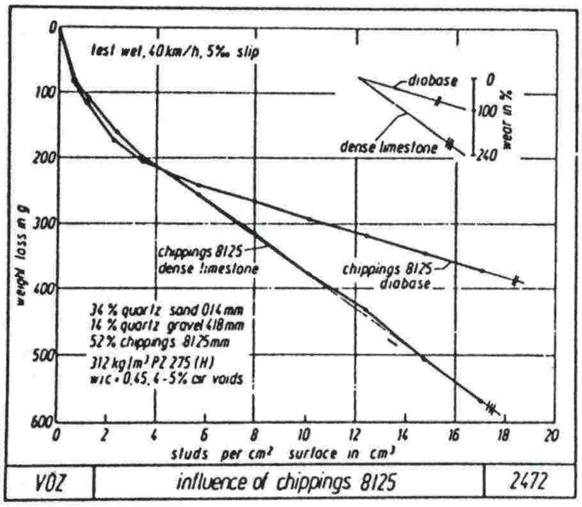


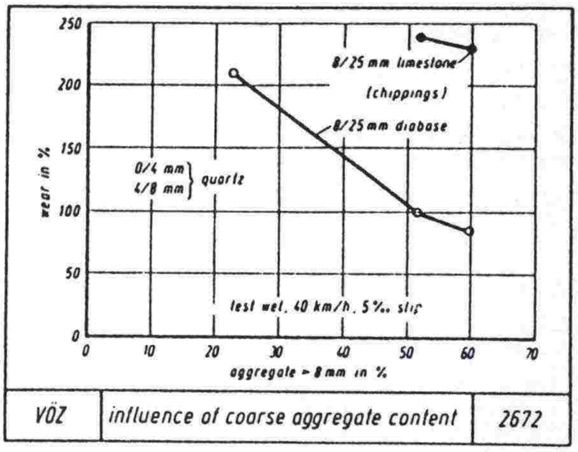
FIGURE B3-11. Wear rates of different pavement types in studded tyre traffic, Minnesota 1970 /1/

in water/cement ratio and powerful vibration improved the wear resistance of concrete, whereas an increase in the amount of cement, did not give better results. It was shown in Austria that the gap in the grading curve, the good quality and the great share of the rough aggregate and the great maximum size of grains are important for a good durability (Figure B3-12). According to the figure the Austrian experience showed that a correct choice of the rough aggregate can result in a fourfold improvement of the wear resistance of the pavement. The increase in compressive strength was proved to be positive but expensive.

Also modified concrete mixes were investigated. It was proved in the United States that a considerable improvement cannot be reached with latex-based additives. Instead, the use of epoxy decreases efficiently the wear, but it causes slipperiness and is very expensive. /1/

At the same time as research activities slackened in connection with the prohibition of studded tyres all over the world, the best results of the foreign investigations were adopted in Finland and wear-resistant concrete was developed in connection with test roads. /18,19,21,22,25,28,29,30,35/





aggregate > 8 mm	wear resistance low : high
wear restistant stone	1 : 2
high content - coarse grading	1 : 1,2
- gap grading 4/8 mm	1 : 1,3
clean surface	1 : 1,2
	1 : 4

VÖZ	concrete of high wear resistance	6671

FIGURE B3-12. The significance of the aggregate quality and grain size distribution for the wear resistance of concrete pavements according to an Austrian investigation /6/

In Finland the following criteria have generally been regarded as characteristical features of the studded-tyre-wear-resistant concrete pavement:

concrete: K50...K60 (91 days)
water/cement ratio:
<0.40
binder: cement and blast-furnace slag in a proportion of 50/50
additives: plasticizer, air entrainment agent
rough aggregate:
crushed rock the mineral composition of which is even, firm and pure, the max. grain size about 32 mm
fine aggregate:
natural sand 0...4 mm
proportioning of aggregates:
a gappy curvature, no grains of 4...8 mm, Figure B3-13, the share of the rough aggregate more than 60 %

The high compressive strength, the low water/cement ratio, the inconstant grading curve and the great share of the rough aggregate have been regarded as important qualifications of the wear resistance. However, the quality of the rough aggregate has a decisive importance to the specific wear of the pavement. When using best pavement aggregates the following value of the specific rutting of the concrete pavement proportioned by the above principles is adopted in Finland:

U_O = 0,12mm/year/1000 ADT

This value is the basic value of the concrete pavement wear in the Finnish design instructions of concrete pavements. It is based on the experiences received so far of rutting measurements in the field and on test track runs made in connection with construction.

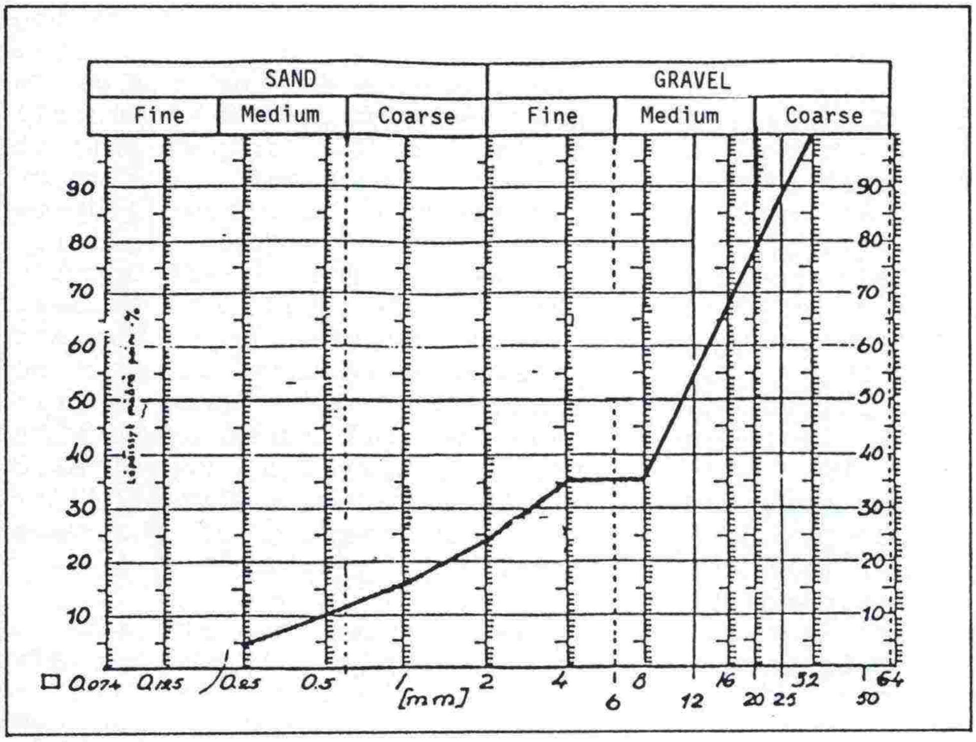


FIGURE B3-13. Gappy design curvature for the aggregate of concrete pavement

A good wear resistance is not only a result of a good concrete mix but also of a successful performance of the work. The aim is to have as much rough aggregates near the surface as possible. An unhomogeneous mix will be revealed as uneven wear. Too high a water/cement ratio and an incorrect spreading method may result in a cement grout layer on the pavement surface, which may wear quickly.

B 332 A new Norwegian development project to further improve the wear resistance of concrete pavements

The wearing problem on heavily-trafficked roads in Norway is - if possible - still more difficult than in Finland. All passenger cars and about 60 % of heavy traffic use studs, the roads are considerably narrow, traffic is concentrated on the few main roads because of the mountainous topography. Concrete pavements have been built as single projects during decades, but concrete has not been used in a wider scale. The latest project was the two-lane concrete pavement of 6,0 km built on the road section of E18 Klinestad-Langåker in the county of Vestfold south of Oslo in the 1970s. Good road concrete of quality class C45 (corresponding K45) was used on this road section; its qualifications correspond to the principles mentioned in Chapter 331. A 2-3 times better wear resistance than that of an asphalt pavement was expected. Rutting measurements on the concrete pavement and on the adjoining asphalt pavement prove that this target has been reached. The rut depth development of the concrete pavement has been approximately 0,28 mm/year/1000 ADT in five years.

Although rutting of the concrete pavement is clearly slower than that of the asphalt pavement, the wear will in the long run result in repair actions also as to concrete pavements. It was discovered that to strengthen their competitiveness in studded tyre conditions the wear resistance of concrete pavements is still to be improved.

An opportunity for this development work aroused in Norway when a decision was made to go on with the reconstruction work of the Main Road E18 with concrete. This took place in 1984 and the 6-km-long road section Klinestad -Tassebekk was paved with concrete in 1986. Meanwhile a wide research programme for the development of even more wear-resistant road concrete was executed in co-operation with the road authorities, contractors and the cement industry.

Wear tests of different concrete mixes made on the wear test track (Figure B3-8) of Norcem Cement formed the basis of the research programme. These tests were completed by other laboratory investigations on concrete and by rutting measurements in the field. The new test track was especially built for this project and the research programme was made out and executed in close co-operation between the different parties. The 1986-concrete pavement was built on the basis of the interesting - even surprising - results of this research project; these results have been published by both the road authorities (Vejdirektoratet) and Norcem Cement /7,5/. The main results of the development project are:

- The compressive strength of the concrete has a decisive significance for wear resistance. An increase in the compressive strength from C45 to C75 decreases the wear by 50-65 %; an even stronger increase in the strength improves the wear resistance even further but not as efficiently (Figure B3-14).

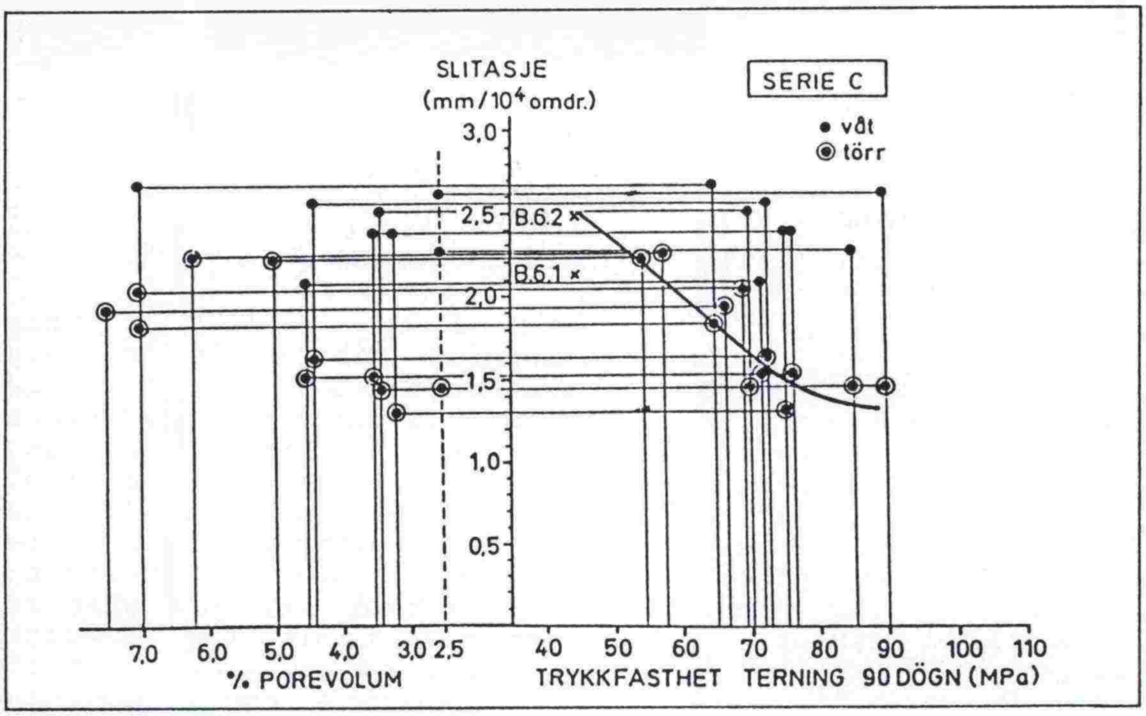


FIGURE B3-14. The relative wear of concrete pavement as a function of compressive strength and air volume according to a Norwegian investigation /7/

- Good frost resistance without air entrainment is achieved with high strength concrete mixes. Air entrainment would reduce the compressive strength by 5 - 6 % per one air volume percent and affects the wear resistance in the negative. Thorough salt scaling frost resistance tests show that frost resistance is excellent without air entrainment /7/.

- The maximum grain size of aggregates is only of little importance as to wear resistance, when it is a question of high strength concrete. A smaller grain size improves the driving comfort of a worn-out pavement and reduces the noise level; thus D_{max} = 22 mm was chosen for the latest pavement and the use of D_{max} = 16 mm is being planned for following projects.

- The gappy grading curve does not result in a better outcome than the constant grading curve.

- The qualities of the rough aggregate finally define the limits within which the wear resistance can be affected. Only the best is good enough for the most heavily-trafficked roads but elsewhere the aimed wear resistance can generally be achieved with local aggregates by adjusting the quality class of the concrete mix.

- Owing to the friction qualities of the worn-out surface such a quality class of concrete should be selected on the basis of wear tests that rough stones remain on the surface as the wear advances. If the wear qualifications of rough aggregates and fine additives are similar in high strength concrete, the pavement may become slippery.

- Also the quality of fine aggregates plays a more important part in the wear resistance than earlier assumed. Crushed sand fraction gives better results than natural sand.

- Wet wear is about three times greater than dry wear and it is not easily affected. Also massive natural granite wears more when wet than when dry.

- Addition of steel or plastic fibres has not significantly improved the wear resistance of the concrete slab.

- The results of the research were directly based on the Klinestad - Tassebekk project when the concrete mix proportioning (Figure B3-15) and the target of the specific wear were defined. The best aggregate (Hornfels) of test runs obtained a specific wear of 0,10 mm/year/1000 ADT (80 % wet/20 % dry) for the C75 concrete, but another aggregate (Syanittprofyr) was, however, used for the job. Its specific wear was measured as 0,14 mm/year/1000 ADT.

The addition of the strength level from C40 to C75 raised the costs of the concrete pavement approximately by 15 %. The rise in construction costs were affected at least by the following factors: development work, wider preliminary tests, a better cement quality and a greater amount of cement, greater demands on the production of the mix, more accurate restrictions of the quality control, better rough and fine aggregates.

STRENGTH CLASS	C75
	kg/m^3
Cement (SP304A)	380
Water	150
V/(C + s) = 0,38	
Sand	742
Crushed gravel 8...16	559
Crushed gravel 16...22	559
Silicate	15
Plasticizer	3

FIGURE B3-15. Proportioning of the Norwegian C75 concrete /11/

The increase in the construction costs is compensated by a more than 2 times higher service life compared with the C40 pavement and by a reduction of maintenace costs to 1/5 of the earlier level as shown by the calculation in Figure B3-16.

Some of the results of this Norwegian investigation may cause objections or they may be in contradiction to experiences of other countries. In any case, the research is a proof of new possibilities in the concrete technology to solve the wear problem in an essentially better way than what is being done today. The research work goes on in Norway and it may accelerate again also in other countries using studded tyres.

B 333 A new Finnish research

Partly inspired by the Norwegian development work it was decided to look for Finnish "super concrete", an even better concrete proportioning as to the wear resistance, in connection with the very state-of-the art project in 1987. The targets of the research were;

1. to find out factors most essentially affecting the wear resistance of conrete pavements and their significance

2. to define an applicable concrete proportioning that is essentially more wear resistant than the present one (ab. 0,05 mm/year/ 1000 ADT)

3. to specify the wear resistance relation between asphalt and concrete

	Rutting speed (mm/1000 ADT)	Service life (years)	Maintenance costs 30 y NOK/m^2	Maintenance costs pres. value NOK/m^2 (by 7 %)
Asphalt	0,50	13	410	134,45
C40 from 1979	0,28	18	220	71,85
C40 offer 1984	0,22	22	190	53,55
C75 cast 1986	0,14	32	60	18,75
C75 (Hornfels)	0,10	41	60	12,85

FIGURE B3-16. Economic survey of the Norwegian C75 concrete based on maintenance costs

The research was carried out by the road and traffic laboratory of VTT under the control of the mutual concrete road group of RWA and RTY. The results of the research have been published in the research report No. 658/1988 of the road and traffic laboratory of VTT.

Due to the conciseness of the research programme - only two test track runs, 6 slabs in both of them - all the information previously available of own and foreign field observations and investigations /7,5,6/ was utilized in the programme. Anyway, the variables to be investigated had to be restricted and it was also decided that each proportioning is represented by one slab only, which, of course, restricts the statistical representativeness of the results. The following variables were included in the investigation (Figure B3-17):

- the compressive strength of concrete (4 different classes)
- the maximum grain size of aggregates (2 sizes)
- the constant and gappy curvature of aggregate
- the quality of the rough aggregate (2 aggregates)
- the use of blastfurnace slag/ silica
- the wearing of the milled surface (1 slab)
- the comparision of the Norwegian and Finnish test tracks
- the wear comparison of rubberized bituminous asphalt with that of concrete

The proportioning data of the concrete slabs are shown in Figure B3-18.

The wear results and wear ratios of the test track runs appear in Figures B3-19 and B3-20.

Generally can be stated that the results of the test track runs are logical and they need not to be explained by a failure in test arrangements. Traditional road concrete (K50 - K 60) wore off 1,3...1,4 times more, the worst concrete (K30) 2,4 times more and traditional asphalt concrete about 3,5 times more than the best concrete quality (K70). In the following the effects of the different variables will be studied briefly on the basis of the research report of VTT.

Slab	Code	Aggregate	Strength	Miscellaneous
1	B32E	Tonalite	K70	Addition of silicate
2	"	"	K60	c/s
3	"	"	K50	"
4	"	"	K30	" AE
5	"	Gabro	K60	c/s
6	B22E	Tonalite	"	"
7	B22J	"	K70	"
8	"the concrete pavement of Norcem"		(K80)	(Addition of silicate)
9=2	B32E	Tonalite	K60	Milled
10	kAB32E	"		Rubberized bitumen
11	AB22E	"		
12	AB22J	"		
13=2	Run in the wear test track of Norcem			
14=4		"		

Clarifications:

Code: B = concrete, kAB = rubberized asphalt concrete, AB = asphalt concrete, E = inconstant aggregate curvature, J = constant aggregate curvature

Aggregate: Tonalite = Very wear-resistant aggregate from Koskenkylä
Gabro = Traditionally wear-resistant Usmin gabro aggregate

Strength: Design strength as an average of the compressive strength

Miscell.: c/s = 50/50 % cement/blastfurnace slag
AE = air entrainment
Milled = About 1 cm was milled of the cast test slab before the wear test run
Also plasticizers are used in concrete mixes
9=2 means that slab 9 is equivalent to slab 2, but milled before the run

FIGURE B3-17. Research programme and variables in a new Finnish investigation /39/

OY PARTEK AB
BTK/Betonilaboratorio Concr. laboratory
KPe/ 04.03.88

Wear test of road concrete
Tiebetonin kulutuskoe (työ 30108)

		30108/1	30108/2	30108/3	30108/4	30108/5	30108/6	30108/7	30108/8	30108/28
design strength 91 (days)	suunnittelulujuus (91 vrk)	K 70	K 60	K 50	K 30	K 60	K 60	K 70	K 80	K 60
performance day	valaistuspäivä	23.06.87	23.06.87	25.06.87	28.07.87	24.06.87	24.06.87	30.07.87	19.08.87	29.07.87
cement	sementtiä kg/m3									
	Pa-Luja (22.06.87) " (n:o 8 "Plattform")	434	197	152	142	200	202	202	390	201
blastfurnace slag	masuunikuonaa "	-	197	152	142	200	202	202	-	201
water	vettä "	116	123	118	156	127	117	100	147	122
aggregate	kiviainesta "	1895	1997	2066	1984	2110	2059	2043	1887	1983
	silika "	28.92	-	-	-	-	-	-	15.40	-
	Scancem SP61 %-sem	4.545	-	-	-	-	-	-	-	-
	Scancem SP62 "	-	0.905	1.563	-	1.250	1.250	1.944	-	0.880
	Melment 10/40 "	-	-	-	0.500	-	-	-	-	-
	Parmix L "	-	-	-	0.040	-	-	-	-	-
	Betokem P "	-	-	-	-	-	-	-	0.789	-
	V/C	0.27	0.31	0.39	0.55	0.32	0.29	0.25	0.38	0.30
settlement	painuma cm	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.0	0.0
stones at the plate	VB, kivet levyssä s	8.0	6.0	8.0	3.5	10.0	7.0	5.0	9.0	6.0
cem. paste at the plate	" sem.liima " "	31.1	33.5	14.2	3.5	22.2	48.7	40.0	-	60.0
air content	ilmapitoisuus %	1.2	0.9	0.8	2.1	0.9	0.9	1.3	1.7	1.2
clensity	tiheys kg/m3									
	7d	2490	2500	2523	2414	2625	2584	2601	2464	2568
	28d	2510	2520	2523	2410	2650	2590	2534	2482	2502
	91d	2490	2487	2496	2427	2623	2596	2564	2460	2520
compressive strength	puristuslujuus MN/m2									
	7d	51.8	37.6	32.7	14.3	45.7	47.5	52.7	66.2	41.2
	28d	65.6	50.5	44.7	23.3	54.6	55.1	62.0	75.6	50.5
	91d	71.4	62.1	54.5	31.1	65.7	66.8	73.5	79.1	59.0
flexural stregth	taiv.vetolujuus (palkit)									
	" " 7d MN/m2	7.39	6.36	4.76	2.26	6.07	6.34	7.55	7.11	5.58
	" " 28d "	8.12	6.76	6.94	3.80	6.50	6.41	9.71	8.38	7.39
	" " 91d "	9.55	8.43	6.56	3.90	8.21	8.94	10.73	8.71	7.07
salt scaling frost resistance test	suolapakkaskoe									
	10 kierrosta, %	0.1	-2.4	-6.6	-11.0	-2.1	-1.3	-0.8	-0.1	-1.9
	25 "	0.0	-3.5	-7.2	-12.6	-3.3	-3.2	-1.6	-0.1	-2.7
	50 "	0.1	-3.9	-8.1	-19.4	-3.5	-3.2	-1.9	-0.2	-3.1
	75 "	0.1	-4.2	-9.4		-3.7	-3.3		-0.7	-4.0
	100 "	0.1	-4.9			-4.4	-3.7		-2.2	
	125 "	-0.1								

FIGURE B3-18. Results of proportioning and examination of materials of concrete slabs /39/

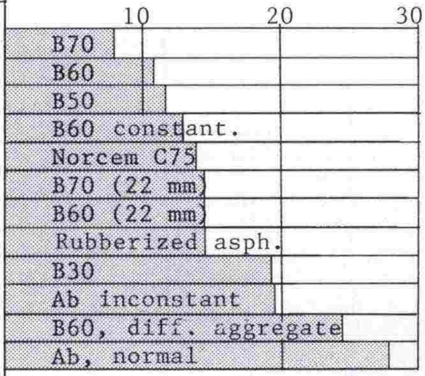
Slab	Code	Wear cm²	SPS g/km	Wear ratio	SPS (g/km/vehicle)
1.	B32E,70,T	6,5	8,2	100	
2.	B32E,70,T	8,5	10,7	130	
3.	B32E,50,T	9,0	11,3	138	
9.	B32E,60,T,J	10,4	13,1	160	
8.	Norcem	11,0	13,9	169	
7.	B22J,70,T	11,5	14,5	177	
6.	B22E,60,T	11,5	14,6	178	
10.	kAB32 T	11,8	14,8	181	
4.	B32E,30,T	15,4	19,5	238	
11.	AB22E, T	15,6	19,7	241	
5.	B32E,60,G	19,2	24,3	296	
12.	AB22J, T	22,4	28,3	345	

FIGURE B3-19. Wear results and average wear relation between the test slabs /39/

1. The significance of the compressive strength

In accordance with the Norwegian research the results prove that the wear resistance can be multiplied by using high strength concrete mixes. The increase in the strength from K30 to K70 meant a 2,4-fold increase in the wear resistance. The test also indicated that concretes of quality class K70...K80 can also be made using cement/slag binders without silica. The concretes used in the test were bad as to their workability, but in practice they can be made suitable for slip-form pavers by using additives. In the test an increase of the compressive strength decreased the initial wear most; the effect was stronger in wet wear than in dry wear.

2. The significance of the aggregate

The results support the opinion that the quality of rough aggregates has a decisive significance also in the wear resistance of concrete pavements. Concrete made with similar proportioning of Usmi gabro wore off 2,6 times more than the same slab made of the Koskenkylä tonalite. Usmi gabro (Hyvinkää) is well-known as a good pavement aggregate and it is a much-used deposit. The characteristic data of the aggregates are:

	LosA	Improved impact value	Abrasion value
The Usmi gabro	17,2	14,1	2,64
The Koskenkylä tonalite	13,7	12,6	1,46

The significance of the aggregate is also emphasized by the fact that a K60 concrete slab made of Usmi gabro wore approximately as much - or even more- as traditional asphalt pavements when the aggregate was tonalite in both cases.

3. The significance of the curvature and the maximum grain size

The test result supports the earlier viewpoint that an gappy grading curvature and a great maximum grain size improves the wear resistance. An increase of the maximum grain size from 22 mm to 32 mm led to a 1,6-fold durability. The choice of the inconstant graining curvature instead of the constant curvature results in a 2,0-fold

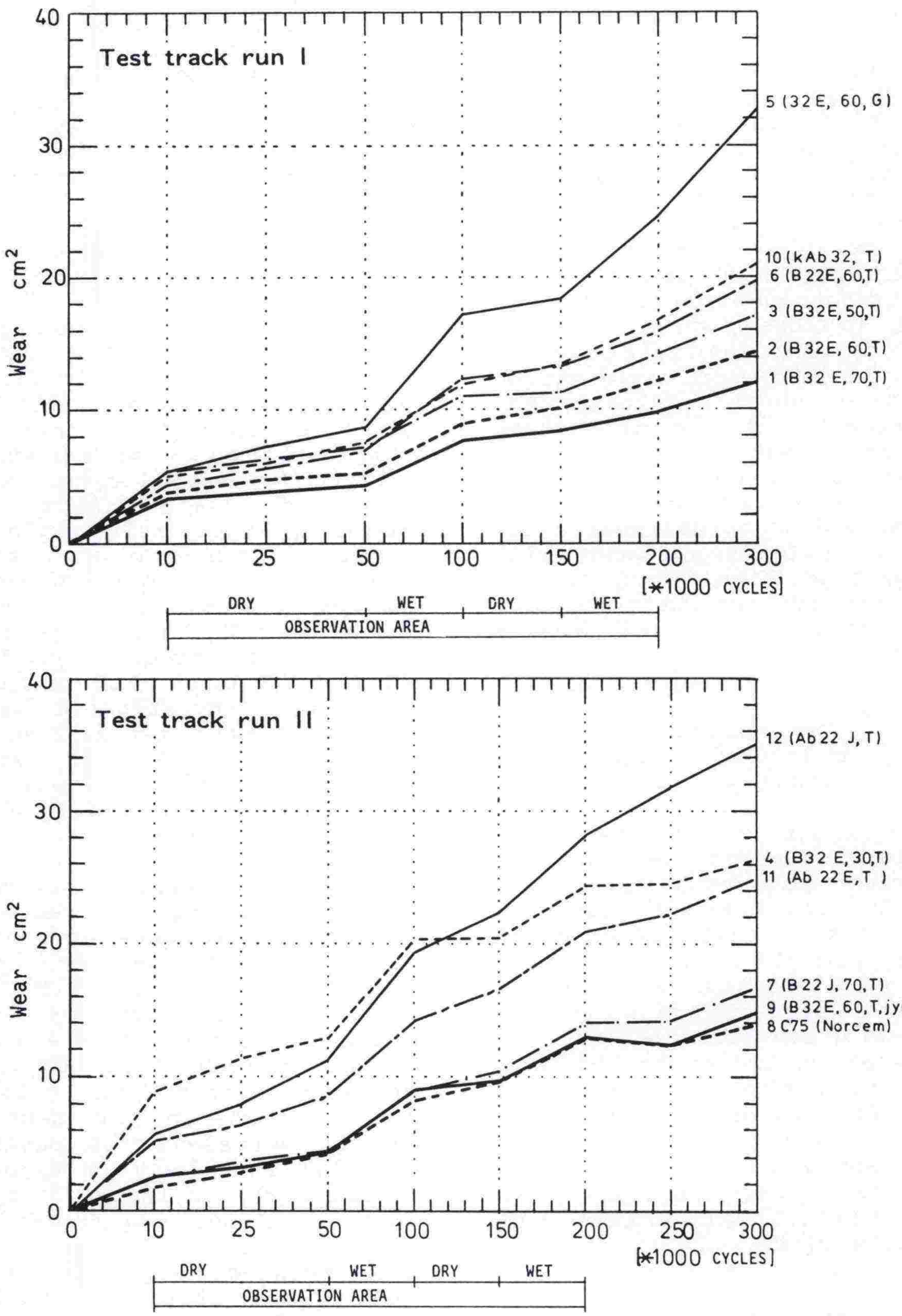


FIGURE B3-20. Wear of test slabs as a function of run cycles /39/

combined effect. The contradiction with the latest Norwegian results is - at least partly - explained by the different function mechanisms of the test tracks. A greater speed (55 - 70 km/h) is applied in the Norwegian track and the rubbing effect has been abolished by conic rings. It is known from other connections that the constant graining curvature is more durable if there is no rubbing. The share of the rubbing is strengthened on the Finnish track (a low speed 32 km/h, a small radius, an upright standard tyre, an eccentric movement of the base) which results in the superiority of the inconstant grading curvature in the test. It remains to be seen which of these test arrangements will correspond to actual wear situations on roads and what will be the final significance of the grading curvature and the maximum grain size for the wear resistance.

4. The significance of moisture conditions (wet/dry)

It is known on the basis of earlier investigations that the concrete pavement wears off clearly more when wet than when dry. In this test the concrete slabs wore off 2,1...2,95 times more when wet than when dry. A better aggregate and a smaller maximum grain size decrease the wet wear according to the test results. This result is, however, based on only one test slab and thus it is not representative. As to asphalt slabs the wet/dry wear ratio was 1,5...1,65, in other words, smaller than with concrete slabs.

5. The effect of milling on the wear

A slab milled in the test wore off 1,23 times more than a corresponding original slab; a wet milled slab wore off 2,8 times more than a dry milled slab. The result shows a greater wear in the beginning before the milling roughness has been evened. On the other hand, the milling has not affected the slab itself in a weakening way as to the wear. However, the milling effects should be investigated more in detail in further studies.

A Norwegian standard slab was included in the test track runs of VTT, it was a C 75 concrete slab made of very strong aggregate (Hornfels), where special cement (SP 30 A) and silicane are used./7/ The standard slab wore off about 1,3 times more than a Finnish K60 slab (cement/-slag 50/50, no silicane), which proves that the Finnish concrete is competitive with the Norwegian concrete also as to wear resistance. The differences are caused by the different qualities of the aggregates. A corresponding proof was obtained when a Finnish K60 concrete slab was investigated on a Norwegian test track: the dry wear of the C75 slab was as high as and the wet wear 1,6 times higher than those of the K60 concrete slab.

The specific wear (g/km/vehicle) and wearing differences of different concrete qualities have been studied with test track runs (Figure B3-19). But to the specific rutting a test track run does not give a direct answer (mm/year/1000 ADT). The specific rutting can only be found out when a slab sawn off from the actual concrete pavement (or cast in connection with the paving work) is included in the test track run. Only by means of this kind of a slab a connection between rut depths measured in the field and the SPS-values of the wear test can be obtained. In this test series the connection was achieved by means of the concrete pavement of the Main Road 12, Villähde-Nastola. In connection with the construction of the concrete pavement in Nastola in 1984 test slabs have been cast and they have been investigated on the test track of VTT in several connections. The latest rutting measurements on the road have been carried out in May 1988. The following ratios have been obtained:

1SPS/on the road = 0.73xSPS/on a test track

1SPS/on the road = 0.0131mm/year/1000 ADT

The specific rutting values for different test slabs have been tabulated on the basis of this calibration in Figure B3-21. The value of

U_o = 0,10 mm/year/1000 ADT

can thus be regarded as a rutting value for the best Finnish concrete pavement.

In this case the quality class of concrete is C70 and the aggregate is very wear-resistant tonalite from Koskenkylä. The values of the specific wear correspond the conditions of the pavement when wet, i.e. 80 %, and when dry, i.e. 20 % of the studded tyre season. This Norwegian way of approach is perhaps even too severe to describe the actual moisture conditions of the pavement in Finland. The specific wear of the best K70 pavement slab when dry would be U_o = 0.047 mm/year/1000 ADT and correspondingly when completely wet U_o = 0.113 mm/year/1000 ADT.

The aimed wear resistance of 0,05 mm/year/1000 ADT in practical conditions was not quite achieved. No doubt, additional investigations can find factors which contribute to an increase in the wear resistance of concrete pavements. One of these factors would no doubt be the washing of aggregates.

The research concentrated on the technical qualifications of the wear resistance, cost issues have not particularly been dealt with. An increase of about 70 mk per cubic metre of concrete in the costs of binders and additives are to be expected when moving from class K30 to class K70 and correspondingly an increase of about 50 mk when moving from class K50 to class K70. A wear resistant aggregate with hauling plays a more significant part in the cost growth than the quality class of concrete.

Slab/Code	Modified SPS-valves (g/km/studded tyre vehicle)		Specific rutting Uo(mm/v/1000 ADT) (=0,0131xSPS/road)
	SPS/test track (80 % wet/20 % dry)	SPS/road (=0,73xSPS test track)	
1 B32E,70,T	10,4	7,5	0,098 *1.
2 B32E,60,T	13,1	9,5	0,124
3 B32E,50,T	14,8	10,7	0,140
4 B32E,30,T	24,8	18,0	0,236
5 B32E,60,G	31.9	23,1	0,303
6 B22E,60,T	18,1	13,1	0,172
7 B22J,60,T	18,0	13,1	0,172
8 Norcem	16,5	12,0	0,157
9 B32E,60,T,Jyr	17,1	12,4	0,162
10 kAB32E,T	17,5	12,7	0,166
11 AB22E,T	22,9	16,6	0,217
12 AB22J,T	32,5	23,6	0,309

FIGURE B3-21. Modified SPS-values and specific rutting of test slabs in the test track run of VTT

B 34 COMPARISIONS OF THE WEAR AND RUTTING OF CONCRETE AND ASPHALT PAVEMENTS

It has become apparent from the above that the wear resistance of the pavement is affected not only by binders but also by many other factors. That is why asphalt and concrete qualities should always be taken into account when studying the wear relation of concrete and asphalt pavements. In fact basic differences of different pavement types can only be studied by building pavements - excluding binders - of as similar materials as possible and by testing wear resistance in similar conditions.' Comparable conditions can be arranged on a test track or by placing the pavements consisting of different binders side by side on the road in similar traffic and environmental conditions. Asphalt slabs proportioned as counterparts with concrete slabs as to the grain size and graining curvature were included in the test series handled in Chapter B 33. When test track results are studied on the basis of these counterparts it can be seen that a bitumen-bound slab wears off approximately 2,0 times more than a cement-bound slab (K60...K70) in dry wear and 1,5 times more in wet wear. Although the values of this "theoretical" wear relation can be considered small and although they will change in different circumstances, this basic relation seems to be correct as to its size range when the wear of similar slabs in standard conditions and in temperatures of 0 - +5°C are investigated. The actual wear relation to the credit of concrete - and especially the rutting relation - is, however, multiplied compared to this. At least the following circumstancial factors affect to the credit of concrete in practice:

- In practice concrete and asphalt are not made with similar grain sizes and with similar grading curvature. A greater grain size of concrete is more profitable as to the wear. In the test series the wear of asphalt concrete Ab 22 was 3,45 times higher than that of the K70 concrete, the maximum grain size of which is 32 mm (Figure 19).
- The rutting of concrete pavements is pure wear; the rutting of asphalt includes deformation of the mix, loosening of stone grains and pressing of stone grains into the mix in high temperatures.
- The rutting of the concrete pavement does not depend on the temperature; the wear of asphalt pavements in a temperature of -25°C is about 10 times higher than in a temperature of 0°C.
- The wear resistance of the concrete pavement can be essentially influenced by an increase in the compressive strength; this is not possible with asphalt.

On the other hand, it is known that the wear of concrete pavements when wet is relatively greater than that of asphalt pavements in general. Furthermore, concrete wears off relatively more than asphalt at high speeds /15/. The relative superiority of concrete is also reduced by the development of asphalt pavements into more wear-resistant than before. The positive effect of modified bitumens and additives and filling materials of asphalt has been known all over the world for a long time. A thorough 5-year investigation, the ASTO project, was started in Finland in 1986; its general objective is an increase in the wear resistance of asphalt pavements by 30 % from the present level.

When looking for numerical values for the wear and rutting relation between concrete and asphalt pavements both foreign and domestic experiences and many previous investigations can be referred to. Then, however, is to be noted that the quality of pavements has not always been analyzed as to the wear resistance; a "standard" asphalt pavement in relation to a "standard" concrete pavement is often used as a criterion.

The investigations of the road administration of Minnesota, utilized widely all over the United States as a basis for decision making in the beginning of the 1970s, resulted in wear relations as shown in Figure B3-11. The numerical values were:

pavement type	wear on a test track (mm/10^6 excesses of studded tyres)	relation
asphalt concrete (standard)	23	3,0
asphalt concrete (high-quality)	19	2,5
concrete (limestone)	12	1,6
concrete (gravel of good quality)	7,5	1,0
concrete (crushed rock)	6,8	0,9

The rutting development of all pavement types were estimated by dividing the wear values of the test track by the figure 5,5 which corresponds the relation of rut widths on the road and in the laboratory.

Rutting measurements before-after have been undertaken together with other research work in Germany in connection with the prohibition of studded tyres in 1975 /23/. The measurements showed that asphalt and conrete of that time wore off in the following relation:

asphalt concrete	3,0
gussasphalt	2,0
concrete pavement	1,0

Wide investigations have been carried out in Austria in the 1970s to improve the wear resistance of the pavements. The wear resistance of the best concrete pavements in relation to the typical asphalt concrete was estimated to be 5:1, even 10:1. In practice, the relation has proved to be smaller, about 3:1, because the variations in aggregates, the unhomogeneousness of the mix etc. also influence the wear of the concrete pavement in field conditions. There are many concrete pavements in Austria and the use of studded tyres have not been prohibited. Thus experiences of the studded tyre wear of both pavement types can still be obtained in Austria, although the utilization degree of studded tyres is diminishing (today about 10 %).

Also concrete pavements were included in a wide Swedish wear research in the beginning of the 1960s /41/. A wear relation of 1,5...2,5 for an old concrete pavement and a standard asphalt pavement was then achieved. It is perhaps due to the little use of concrete pavements that additional investigations on concrete were not made.

The studded tyre wear has been intensively studied in Norway since the beginning of the 1960s, although concrete and asphalt pavements have not been included in same investigation programmes in Norway. Wear patterns of asphalt pavements have been found out on the basis of long-term rutting measurements and concrete pavements have been investigated both in the field and laboratory. The rutting measurements made on the Main Road E18 led to the beginning of the development project in 1984: the asphalt pavement wore off 2,5...3 times more than the concrete pavement (C45 from 1979). The C75 concrete was developed during the project;

its wear resistance is 2 times higher than that of the C45 concrete. Thus the new wear relation of concrete and asphalt is 5,0 -6,0/1 in Norway. The significant results of the latest Norwegian development work have been described in Figure B3-22. A value of 0,43 mm/year/1000 ADT is used as a rutting speed of asphalt pavements in the pavement design specifications of the Norwegian road administration. The value varies per province, thus Vestfold, where most of the Norwegian concrete pavements have been built, uses a value of 0,50 mm/year/1000 ADT in comparisions for asphalt. When 0,10...0,14 mm/year/1000 ADT was obtained as a specific wear for the C75 concretes, the wear relation will be 3,0...5,0/1 when these average specific wears of asphalt pavements are used.

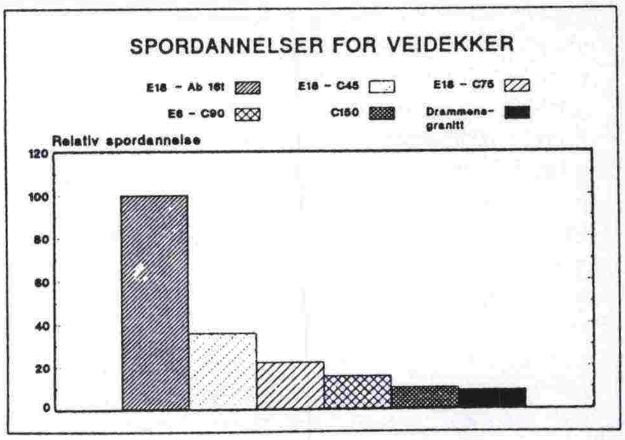


FIGURE B3-22. Wear ratios in a Norwegian investigation (rutting of C45...C150 concrete and solid granite in relation to Ab16 asphalt) /16/

As in Sweden and Norway the studded tyre wear is the main problem of the pavement policy also in Finland. The rutting of asphalt pavements have been studied for more than 20 years both in laboratories and with field measurements. The pavement design instructions of the Road and Waterways Administration /27/ are mainly directed at the control of the wear problem in the pavement design. However, the instructions have not led to any improvement in the situation, but the studded tyre wear is constantly an "open wound" in the road management in the conditions of growing traffic. The research programme, the ASTO project, started in 1986 tries to find improvement to the situation. The pavement design instructions based on field measurements indicate that the specific wear on asphalt roads is 0,3...0,7 mm/year/1000 ADT when ADT > 2000 and 0,5...1,0 mm/year/1000 ADT when ADT < 2000. These values are not considered to include deformation of the base, which is 10...20 % of the total rutting even on best asphalt-paved roads and may be even greater than the wear on weakly-load-bearing roads (<200 MN/m^2). Based on rutting measurements it is further stated that the specific rutting is greater with small traffic flows than with great traffic flows. Even a small traffic flow - 1000 ADT - is enough to rut the asphalt pavement aspproximately 1,7 mm/year, but an increase up to 10000 ADT in traffic flow causes rutting of only 0,38 mm/year/1000 ADT. This results from the fact that lightly-trafficked roads are narrower and also from deformation of asphalt in summer temperatures.

The rutting of concrete pavements have been studied with rutting measurements also in Finland. These measurements have been carried out on all concrete pavements (Figure B3-23) built in the last decades /22,24,21/. An annual increase in the rut depth of asphalt pavements has been 2,3...5 times more than on concrete pavements. Old concrete pavements show specific wears of 0,16...0,18 mm/year/1000 ADT. Newer concrete pavements (K50...K60) have reached specific wears of 0,12 mm/year/1000 ADT, which is the basic value of the Finnish design instructions of concrete pavements. When the above wear relation of 2,3...5,0

Test area		Rut depth Years	Tot.rut. mm	Rut mm	ADT 1)	Specific rutting mm/1000 ADT/	Rutting ratio Ac/Concr.
Mr 2 Palojärvi-Olkkala (8,9,10)	Ac	75...77	5,0	1,9	2780	0,36	2,4
"	Concr.	"	3,6	0,8	"	0,15	
"	Ac	78...82 2)	5,2	5,2	3530	0,40	2,3
"	Concr.	75...82 2)	6,5	3,7	"	0,18	
Hw 180 Ylikylä-Parainen(11)	Ac	76...79 3)	5,0...13,0	5,0...13,0	6230	0,58...0,86	n. 4...5
"	Concr.	71...81	7,0...22,6	7,8...11,8	1800÷6230	0,14...0,17	
Mr II 50, radial r. III (12)	Ac	78...81	15,8...27,1	15,8...27,1	24 000 4)	0,22...0,38	ka. 3,0
"	Concr.	76...81	16,8...29,4	6,8...15,1	"	0,06...0,12	

1) ADT of the latest measuring year.
2) With a rut device added with a rut depth of ± 10 %
3) TVL/Turku district straightedge measurement
4) Medified of ADT 20 000 of one carriageway taking account the different traffic flows of the lanes in relation to right/left = 60/40 %.

FIGURE B3-23. Rut depths and wear ratios of Finnish pavements according to measurements of VTT /25/

is taken as an intial value and when - carefully estimated- 1,5-fold wear-resistant concrete has been found in the test series of Chapter B 333, a conclusion can be drawn that the present standard asphalt pavements wear off 3,5...7 times more quickly than the Finnish "super concrete pavement" (K70 concrete, tonalite as an aggregate). A rutting relation of 2,5...3,5/1 is obtained compared with asphalt pavements built of the same strong aggregates.

B 35 SUMMARY

The wide use of studded tyres in the 1960s revolutionized the road pavement policy in all cold countries. The rutting of pavements became a dominating problem - however, only temporarily in most countries. Studded tyre traffic was proved to be disastrous for all pavements and after public discussions the use of studded tyres was prohibited in most western countries in the mid-1970s.

Studded tyres and wear of pavement seem to be a permanent part of the traffic culture in Scandinavia; according to the public opinion studded tyres mean security in the winter traffic.

When reconstruction of asphalt pavements due to studded tyre wear costs hundreds of millions of marks every year a reduction of wear defects is an object of a constant struggle in Finland and in the rest of Scandinavia. Wear effect of studs have been decreased by restricting stud force and protrusion. Introduction of sleeve studs (security studs) to the market raises hopes of further decrease in the wearing stress. Wear qualities of asphalt pavements have been continuously studied and a better wear resistance have been achieved. The use of rubberized bitumen is the latest invention in the asphalt field; it contributes to an improvement of wear resistance and stability but simultaneously to a rise in costs.

But in practice the detrimental wear of asphalt pavements is still an "open wound" (Figure B3-24). Hopes of a better future have been concentrated on the 5-year investigation programme- the ASTO project -by means of which an improvement of 30 % in the wear resistance will be aimed at.

It has always been a well-known fact that concrete pavements are more wear resistant than asphalt pavements (Figure B3-25). A 2,5...3,5-fold durability has been reached in conformity to results of many countries; this is entirely of another size range than the durability of asphalt. The latest Norwegian and Finnish investigations have introduced high strength concrete mixes on road pavements which raise the wear resistance to another order of magnitude altogether. <u>New high strength concrete mixes show still a 2,5...3,5-fold wear resistance compared with asphalt pavements built of the best and same aggregates, but compared with the present Finnish standard asphalt pavements the new high strength concrete (K70, tonalite as an aggregate) offers a 3,5...7-fold wear resistance.</u> This means that a pavement which has been repaired at intervals of 2...3 years, endures 10...15 years with no need for rutting repairs if it is paved with concrete. Usual traffic flows (< 15000 ADT) do not cause rutting repairs at all during the service life of the pavement. When ruts begin to appear they are repaired by diamond grinding, which method is a widely developed routine method in many countries.

Studded tyre traffic is a threat also for the service life of the concrete pavement. Adaption of the latest concrete technology has changed this threat into a challenge. A concrete pavement can be and has been designed so wear-resistant that studded tyre wear does not essentially decrease the efficient service life of the pavement. A constant good service level can be prevailed by grinding the possible ruts before they become a significant inconvenience to traffic.

Being prepared for wear and increasing the wear resistance of the pavement cause additional costs in both asphalt and concrete pavements. The additional costs of concrete pavements are composed of a higher mix quality, consideration of the grinding margin in the slab thickness and of grinding costs. These costs are compensated by a long service life of the concrete pavement and by a good service level also in exceptional traffic conditions caused by studded tyre traffic.

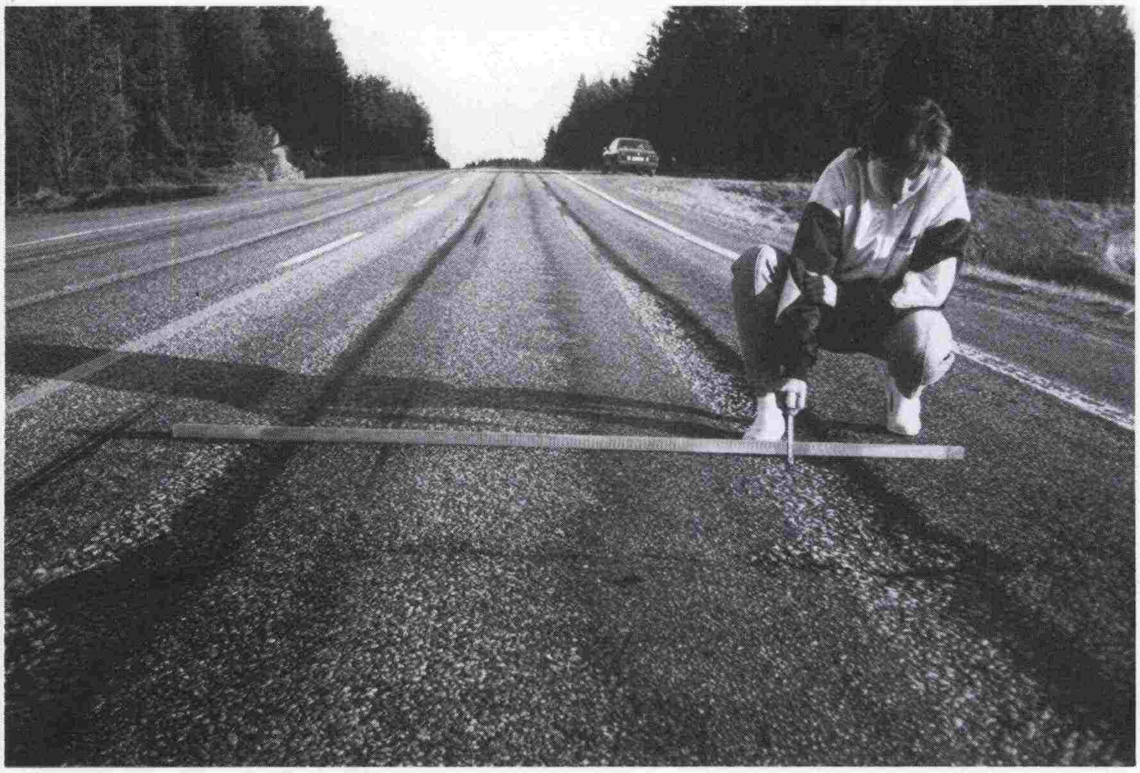


FIGURE B3-24. A rutted asphalt pavement, spring 1988 (mr II 41, Pöytyä, ADT 2200, rut repaired section)

FIGURE B3-25. A rutted concrete pavement, spring 1988 (Skräbbölentie, Parainen, ADT 2000, built in 1958)

CHAPTER B3 CONCRETE PAVEMENT WEAR DUE TO STUDDED TYRES

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10. Betong for ny E-18 gjennom Vestfold etter anbud på alternative dekkeløsninger, Våre Veger, nr 9/1985

11. Omfattande forsøksprogram gjennomført i Vegslitern, H.A. Ramsvik, Våre Veger nr 8/1986

12. Betong eller asfalt for ny E-18 gjennom Vestfold, Ivar Wulff, Våre Veger nr 9/1984

13. Nytt betondekke på E-18 i Vestfold/Vedlikeholdsmessige betraktninger avgjorde valget, Erik Andersen, Våre Veger nr 8/1986

14. Produksjon og Fremstilling av Høyfast Betong, Oistein Dyblie, Norsk Betongdag, Oslo, 1986

15. Slitestarke betongdekker for veier/aktuelle erforinger fra utprøving i Veisliteren, N. Pedersen, NVF, utskott 32, temamöte i Åbo, 1987

16. Høyfast betong overlegen som vegdekke, Våre Veger nr 4/1988

17. Betong-Nils, Portrett, Våre Veger nr 4/1988

18. Betonipäällystekokeilu Kehä III:lla, yhteenveto vuosilta 1971 - 1976, VTT/Tie ja liikenne, tutkimusselostus 71, Espoo 1977

19. Betonipäällystekokeilu Kehä III:lla, vuoden 1979 mittaustulokset, VTT:n tutk.sel. 169, Espoo 1979

20. Betonipäällysteiden teknillisistä ja taloudellisista käyttöedellytyksistä Suomessa, VTT Tie ja liikenne, tutk.sel. 184, Espoo 1980

21. Palojärvi - Olkkala -koetie, yhteenveto vuosilta 1973...78, VTT Tie ja liikenne, tutki.sel. 161, Espoo 1979

22. Betonipäällystekokeilu Kehä III:lla, jälkiseurannan tulokset v. 1981, VTT Tie ja liikenne, tutk.sel. 285, Espoo 1982

23. Asfaltti- ja betonipäällysteiden kulumismekanismi ja kulutuskoeradat/Matkakertomus opintomatkasta Länsi-Saksaan ja Sveitsiin/Anssi Lampinen, Espoo 1981

24. Paraisten betonipäällysteen tutkimus, VTT Tie ja liikenne, tutk.sel. 298, Espoo 1982

25. Asfaltti- ja betonipäällysteiden kulutuskestävyysvertailu, VTT Tie ja liikenne, tutk.sel. 407, Espoo 1983

26. Kehä III:n betonipäällysteen jyrsintäkokeilu vuonna 1983, VTT Tie ja liikenne, tutk.sel. 407, Espoo 1983

27. Päällystesuunnittelu 1984, TVH 742853, Helsinki 1984

28. Betonipäällystekokeilu Kt 45:llä välillä Rajaniemi - Lakalaiva 1983, seurantatutkimus 1984, VTT Tie ja liikenne, tutk.sel. 474, Espoo 1985

29. Päällysteiden kulumiseen vaikuttavat tekijät, VTT Tie ja liikenne, tutk.sel. 486, Espoo 1985

30. Palojärvi - Olkkalan betonipäällysteen loppuraportti, VTT Tie ja liikenne, tutk.sel. 476, Espoo 1985

31. Nastarenkaat ongelmana, Matti Sistonen, Tie ja niikenne no 12/1986

32. Nastarengaskulutuksen alentaminen, Matti Sistonen, Tierakennusmestari 4/1985

33. Päällysteiden kuluminen - ratkaisematon ongelma, Matti Leinonen, Tierakennusmestari 4/1985

34. Nastojen ja talvikunnossapidon vaikutus Uudenmaan piirin toteuttamaan päällystepolitiikkaan, Kari Alastalo, Tierakennusmestari 4/1985

35. Betonipäällystekokeilu VT 12:lla ja Kt 45:llä, seurantatutkimus 1986, VTT Tie ja liikenne, tutk.sel. 603, Espoo 1987

36. Tiepäällystekiviaineksien kulutuskestävyydestä ja testaamiseen soveltuvista koemenetelmistä, TVH 743952, Helsinki 1987

37. Tiepäällystöiden työselitys 1988, TVH 742802, Helsinki 1988

38. Kestopäällysteet, esiselvitys Pank ry, Espoo 1988

39. Betonipäällysteiden kulumisominaisuuksien selvittäminen, VTT Tie ja liikenne, tutk.sel. 658, Espoo 1988

40. Untersuchung des Kraftschlusses auf winterlichen Strassen unter berücksichtigung der Strassenschonung, D.K. Rouyren, Köln 1985

41. The Effext on Concrete Pavement of Studded Winter Tires, B. Örbom, 1st European Symposium on Concrete Pavements, Paris 1969

CHAPTER B4
CONCRETE ROADS IN SEVERE CLIMATES

CHAPTER B 4

CONCRETE ROADS IN SEVERE CLIMATES

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B 4
CONCRETE ROADS IN SEVERE CLIMATES

B 40 INTRODUCTION

In a country of seasonal frost action like Finland concrete pavements are exposed to circumstances where frost action of subgrade - differential frost heave and decreased spring bearing capacity - dictates to a great extent the design requirements and durability conditions of the road construction. Construction of rigid concrete pavements in these conditions arouses many questions:

- Does the rigidity of the pavement contribute to a reduction of thickness of the pavement structure?
- Can the rigidity and joints damage the service level and long-term durability of the road due to frost action?
- How do frost-susceptible circumstances affect the design of the pavement slab?
- How have concrete pavements built for frost-susceptible conditions endured?

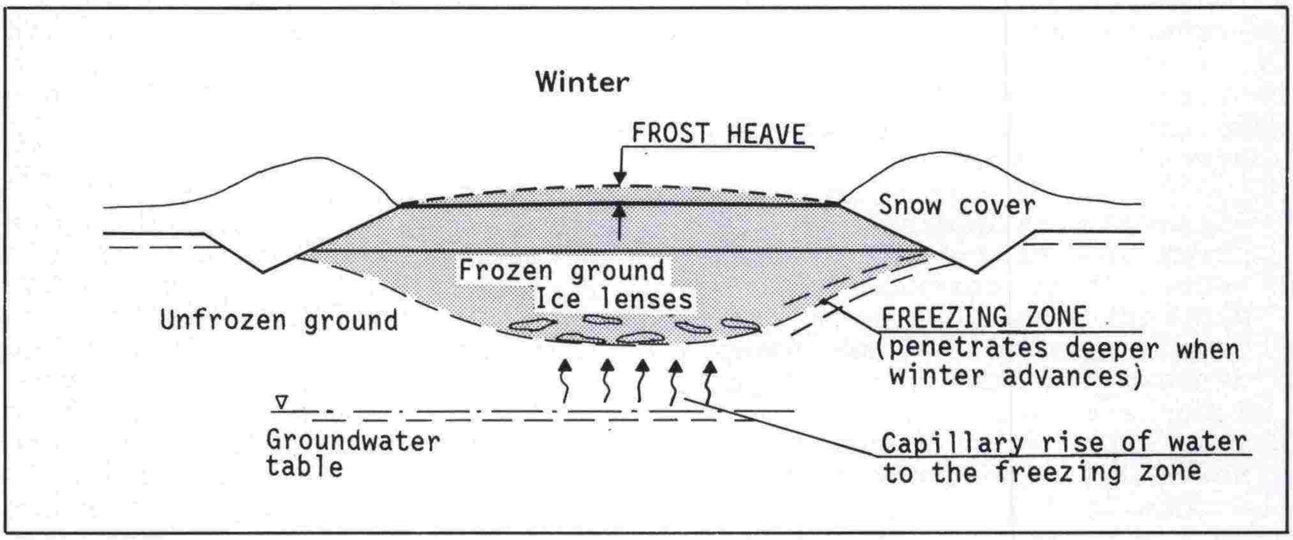
Finland has both positive and negative experiences of the durability of concrete pavements built in our severe climate. However, because of the relatively little experience of our own the intention of this report is to find additional information to the above questions. Finnish and foreign literature and research journeys in seasonal frost countries as well as interviews made there have contributed to shedding new light on the matter. Because frost action plays an important part in the structural design of roads in severe climates like in Finland, also basic facts related to frost action are described in the following.

B 41 BASIC FACTS ABOUT FROST ACTION

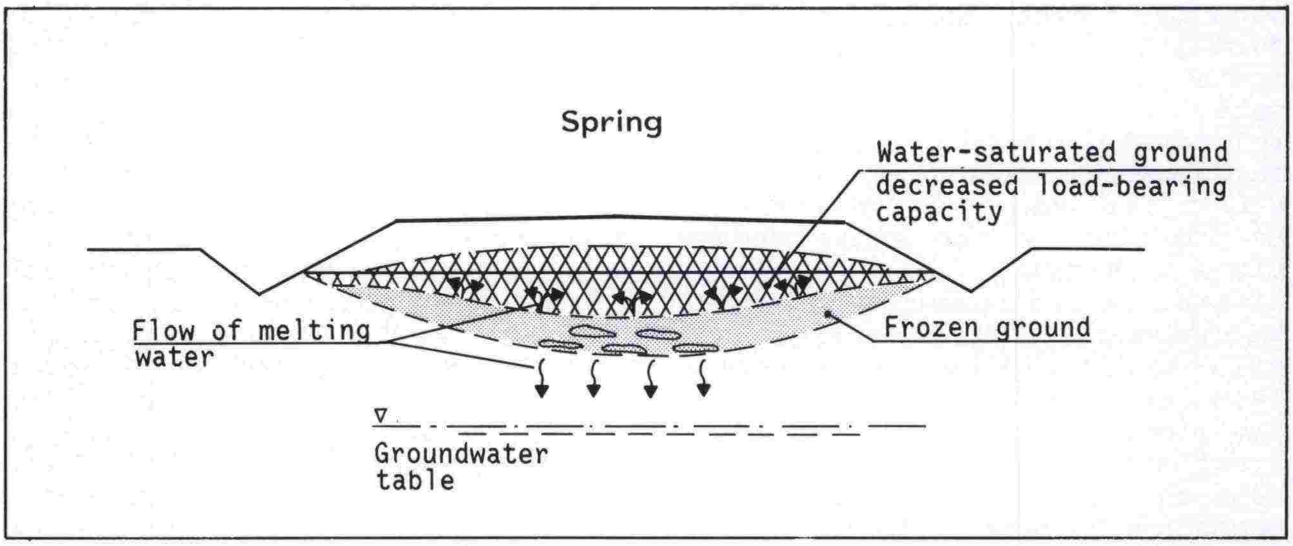
B 411 Frost action

When air temperatures fall below zero, the ground starts to freeze. The longer the frost seasons last and the more severe they are the deeper the frost penetrates. When water in the ground freezes, the strength of the soil increases. Due to the freezing, water expands and fills the voids in the soil, so the freezing as such does not produce a considerable growth in volume nor a rise of the pavement surface. Frost action, which is here defined as a negative phenomenon, occurs when free water collects in the freezing zone and freezes to form ice lenses in the course of a long frost season, Figure B4-1. In soils rich in fine particles water freezing into lenses can be water which is rising capillarily to the freezing zone or water flowing at the freezing zone. Formation of ice lenses means that the ground level is rising. This phenomenon is called frost heaving and it will be the greater the deeper the frost penetrates or the longer the frost season lasts, /38/.

When air temperature rises above 0°C the ground starts to melt. If temperatures remain low for a longer time, melting will proceed from the bottom upward influenced by the heat of the earth. In this case water escaping from ice lenses can join groundwater and no problems will arise. If the temperature rises rapidly - as is often the case in spring in Finland - the melting will occur from the top downward. Water escaping from ice lenses is unable to meet groundwater and it will saturate melting soil courses, Figure B4-2. An increase in the load results in an increase in the pore-water pressure and the melting soil will loose most of its load-bearing capacity. A decreased spring bearing capacity together with frost heave is typical of frost action, Figure B4-3.



B4-1. Frost action of the road, winter



B4-2. Frost action of the road, spring

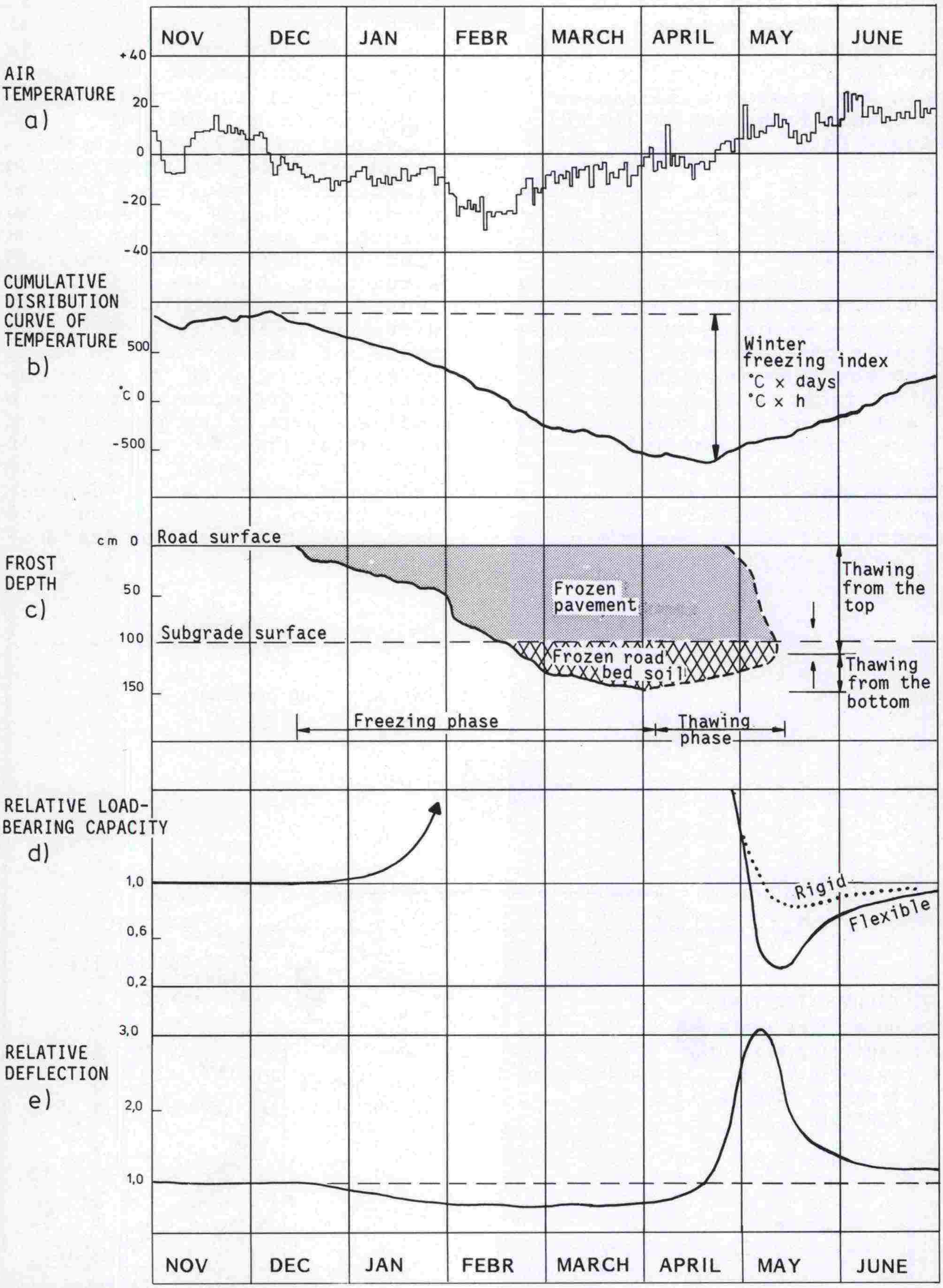


FIGURE B4-3. Variations of temperature, freezing index, frost depth and load-bearing capacity in seasonal frost action areas

B 412 Prerequisites for frost action

The following three conditions must be present simultaneously to enable frost action to occur, Figure B4-4:

- there is frost-susceptible soil at the depth of frost penetration or immediately underneath
- the groundwater table is so high that the capillary rise to the freezing zone is possible or free water is flowing at the freezing zone
- the frost season is so long and severe that formation of ice lenses is possible

The severity of frost action is a combined effect of these factors. If one factor is missing no frost action will occur, so that when affecting one of these conditions the entire frost action can be affected.

The depth of frost penetration and the magnitude of frost heave and spring bearing capacity can be estimated by investigating frost action conditions beforehand; this helps to design the structure so that frost action does not cause damage to road structures. The sensitivity to frost action can be defined for different soil types on the basis of the content of fine particles (< 0,02 mm) in the soil. The groundwater table is defined with piezometers, and the duration and severity of the frost season i.e. the freezing index, as a degree-hour ratio (0°C x h) on the basis of temperature measurements.

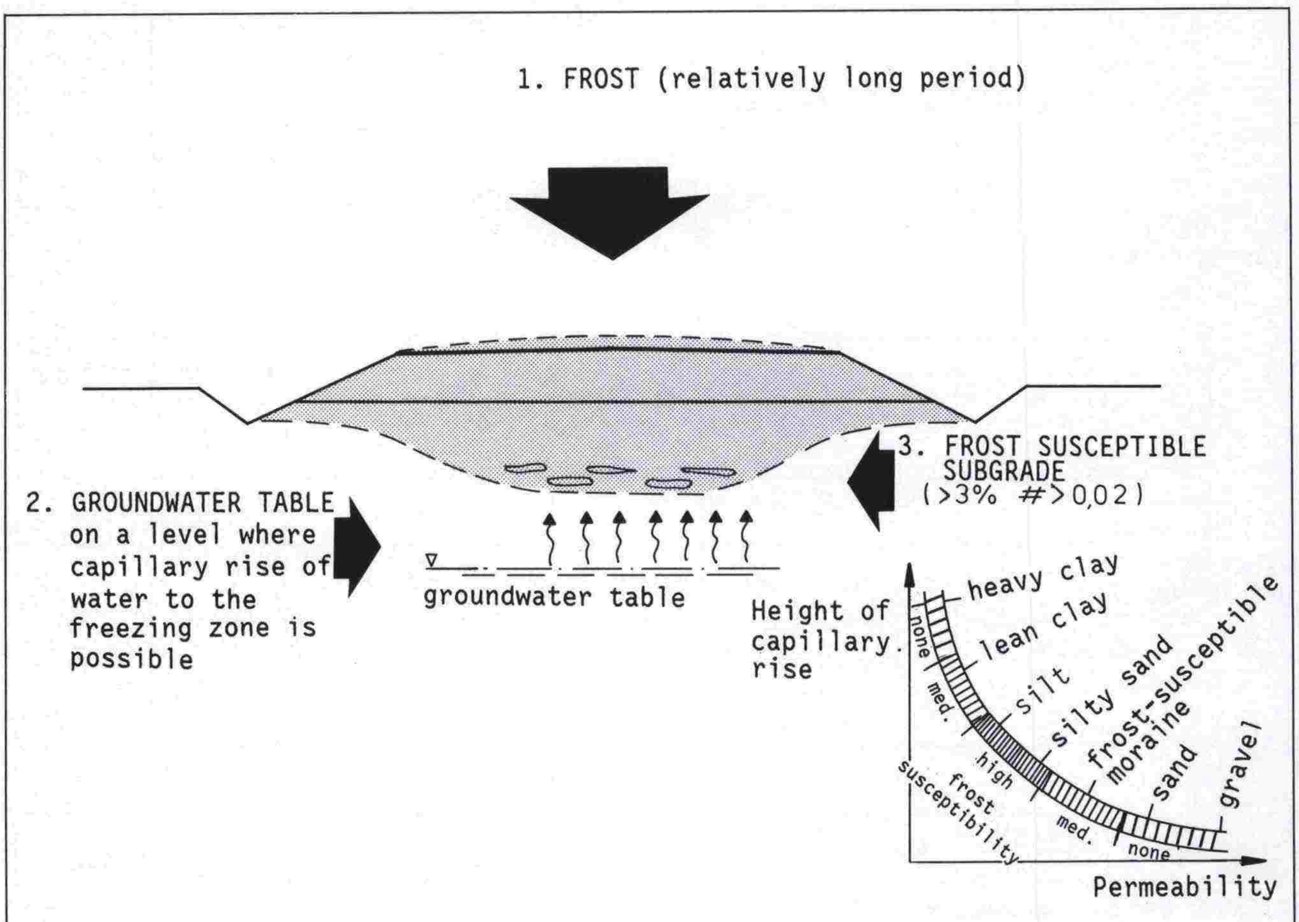
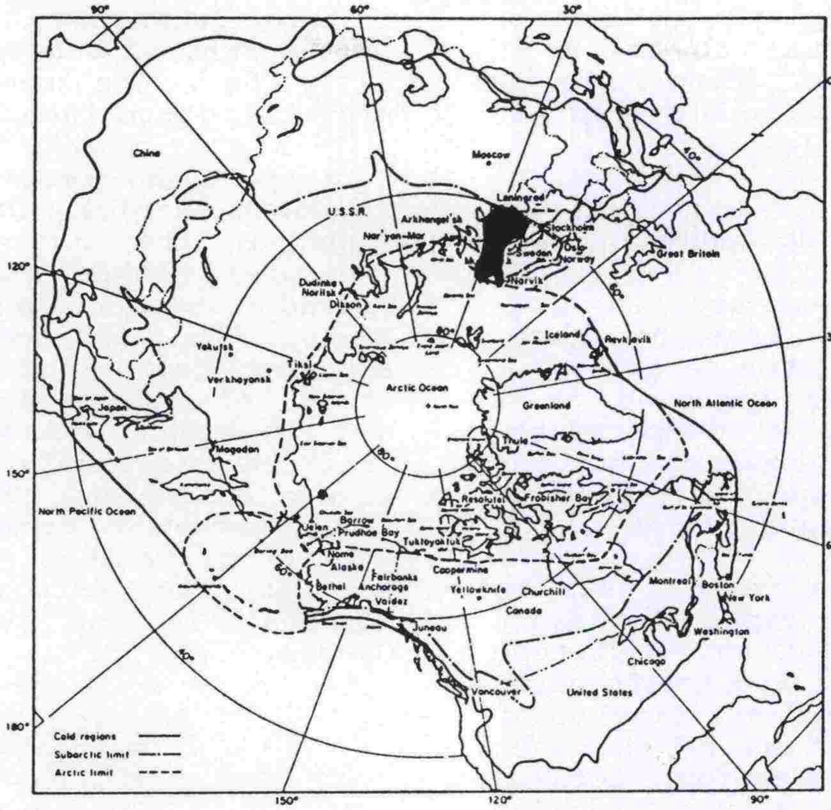


FIGURE B4-4. Prerequisites of frost action



- MILD CLIMATE: The average temperature of January >-3 °C, <0 °C
- SUBARTIC: The average temperature of January <-3 °C; mostly 4 months > 10° C average temperature
- ARCTIC: The average temperature of July < 10° C

Finland belongs to most northern subarctic regions

(/COLD REGION STRUCTURAL ENGINEERING, E. ERANTI, G. LEE)

B4-5. Cold areas of the Northern hemisphere

B 42 FROST ACTION CONDITIONS IN FINLAND AND IN OTHER SEASONAL FROST COUNTRIES

B 421 General climatic charecteristics

Finland belongs to a subarctic climatic zone, Figure B4-5. General characteristics of the climate are /20/:

- The yearly average temperature -2...+5°C (in different parts of the country)
- a considerably cold winter, the average temperature of January -5...-15°C
- a considerably warm summer, the average temperature of July +13...+17°C
- a considerably low amount of rainfall, 400...700 mm/year
- seasonal frost action, the average freezing index 500-2000°C x days/year.

Inspite of the northern location Finland enjoys a relatively temperate climate thanks to the Gulf Stream; permafrost generally prevails on the same latitudes in Siberia, Canada and Alaska whereas frost melts from every part of the country in Finland every year. (It is true, however, that there are local palsa formations comparable with permafrost in Lapland). Climatic conditions equal to those in Finland can be found only in Central and Northern Sweden and in Soviet Karelia.

B 422 Freezing index

Frost conditions are generally described as a freezing index, which shows the severity of winter and is measured as a result of daily temperatures and of the duration of the freezing season, °C x days or °C x hours (Figure B4-6).

Freezing index values help to evaluate the depth of frost penetration and the magnitude of frost heave; thus the freezing index is a good starting point in the estimation of the need for frost protection. Because variations in annual freezing indices are great even in the same district, a probability to which the structures are dimensioned is to be selected. It is very common in seasonal frost countries that the greatest freezing indices occurring once in ten years are used for the design, Figure B4-7, Figure B4-8, although the severity of winters is also described as an average freezing index (e.g. during 30 years), Figure B4-9, Figure B4-10), /20,3/

From the point of view of freezing indices Finland is one of the coldest seasonal frost areas in the world. A continuous long freezing season when frost can penetrate deep is typical. Also a short and violent spring with total frost melt is a typical feature. As to freezing indices only Central and Northern Sweden and Estern Karelia in the Soviet Union have corresponding conditions in Europe. There are severe frost areas also in the mountains in Norway and in the Alps in Austria and Switzerland, but more locally than in Finland. At least Switzerland, Austria and Finland have built concrete pavements in these areas. Freezing indices in the rest of Europe are generally only one tenth of the prevailing conditions in Finland.

Corresponding freezing indices in North America can mostly be found in the surroundings of the Great Lakes both in the United States and Canada, Figure B4-10, /3/. Concrete pavements are most commonly built in the states of Minnesota, Wisconsin and Michigan in this area. Concrete pavements in more severe conditions can be found in the surroundings of Winnipeg, capital of Manitoba, in Canada and in the surroundings of Montreal in the province of Quebec.

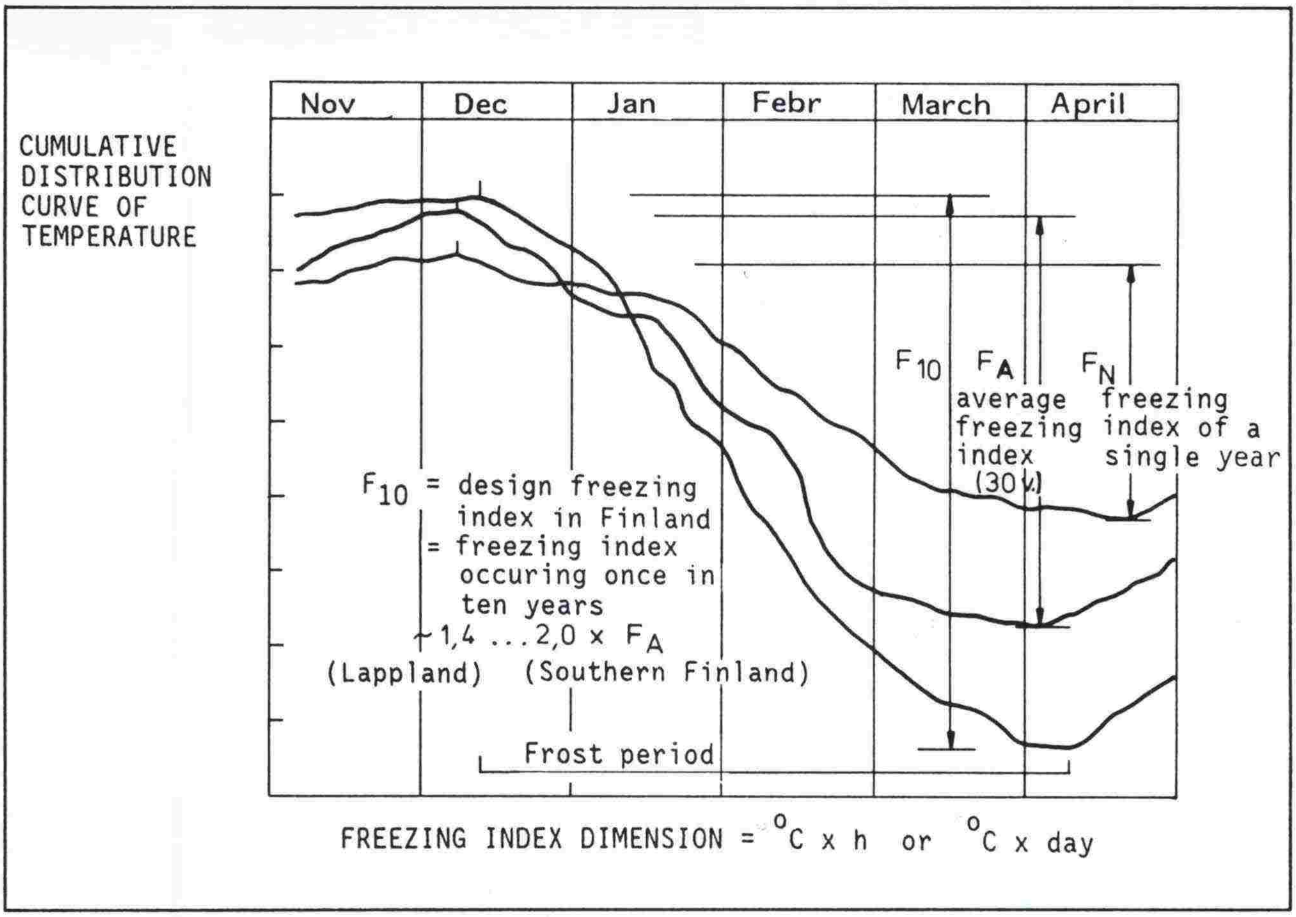
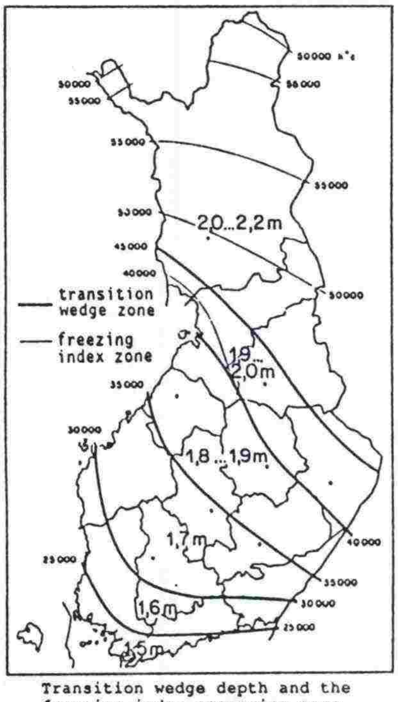
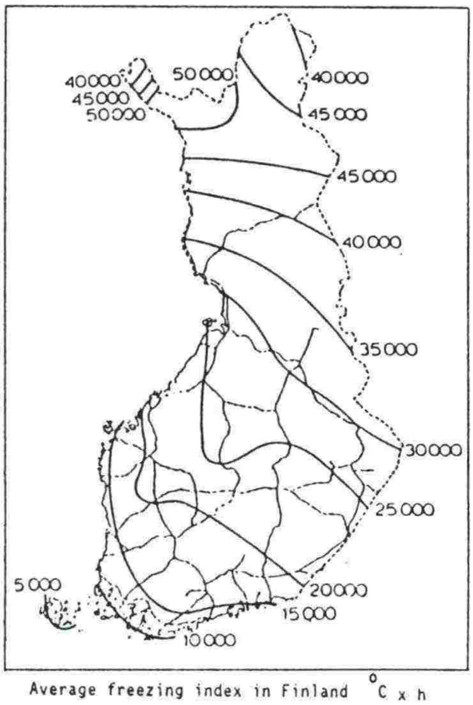


FIGURE B4-6. Different concepts of freezing index



Transition wedge depth and the freezing index occurring once in ten years. The depth of the wedge made of gravel or crushed stones is 0,2 m deeper and that made of blasted stones 0,5 m deeper (TVH, Road structure, 3.2-1)

FIGURE B4-7 a.
Transition wedge depth and the freezing index occurring once in ten years in Finland /16/



Average freezing index in Finland °C x h

FIGURE B4-7 b. Average freezing index in Finland (°C x h) /20/

	F_{-100}	F_{-50}	F_{20}	F_{10}	F_{5}	F_{2}	F_{5}	F_{10}	F_{20}	F_{50}	F_{100}
Turku, lentoas.	0	700	2500	4200	6600	12300	19800	24800	29700	35900	40500
Helsinki, Kaisaniemi	0	650	2300	3900	6100	11300	18300	23000	27400	33100	37400
Kotka	2200	3400	5200	7000	9500	15200	23000	28200	33100	39500	44300
Jokioinen	3300	4300	6100	7900	10300	15800	23400	28400	33100	39300	44000
Heinola	4900	6000	7900	9700	12100	17900	25600	30700	35700	42000	46800
Vaasa	2300	3500	5400	7400	10000	16200	24500	30000	35300	42200	47300
Jyväskylä	6500	7600	9500	11300	13900	19800	27700	33000	38100	44600	49600
Mikkeli	7700	8800	10700	12600	15100	21800	29100	34500	39600	46200	51100
Kuopio	10400	11500	13400	15200	17700	23500	31300	36500	41500	47900	52800
Oulu	10600	11800	13800	15700	18400	24600	33000	38600	43900	50800	56000
Kajaani	13300	14500	16500	18500	21300	27700	36300	42000	47500	54600	60000
Rovaniemi	19300	20700	22900	25000	27900	34800	44000	50200	56100	63800	69500
Kuusamo	21600	22900	25000	27000	29800	36300	45100	50900	56500	63700	69100
Sodankylä	24000	25300	27400	29600	32500	39300	48400	54500	60300	67900	73500

FIGURE B4-8. Annual freezing indices of various towns with different probabilities in Finland

B 423 Soil and groundwater conditions

Scandinavia is one of the last northern areas released from under the inland ice. The basement rock is old, strong and non-weathering, there are plenty of exposures of rock and loose soils are relatively shallow. The main soil types are moraines and there are also clay soils on the coast. Northwest-southeast esker formations are typical. There are plenty of lakes and moors. A general feature of the topography is relatively low, but varying. The groundwater table is high although the annual amount of rainfall is rather low (400...700 mm/v). The Finnish soil and groundwater conditions are liable to frost action.

The North American topography is much flatter in the seasonal frost areas than in Northern Europe. There are not many rocky areas, loose soils are generally thick moraine and silt deposits. Wide, low prairies extend also to northern areas. The groundwater table may be high and the conditions are prone to frost action.

Corresponding local soil and groundwater conditions to those in Finland can be found in the Swiss and Austrian Alps, but, in practice, frost action conditions differ in Finland because of the deviating topography.

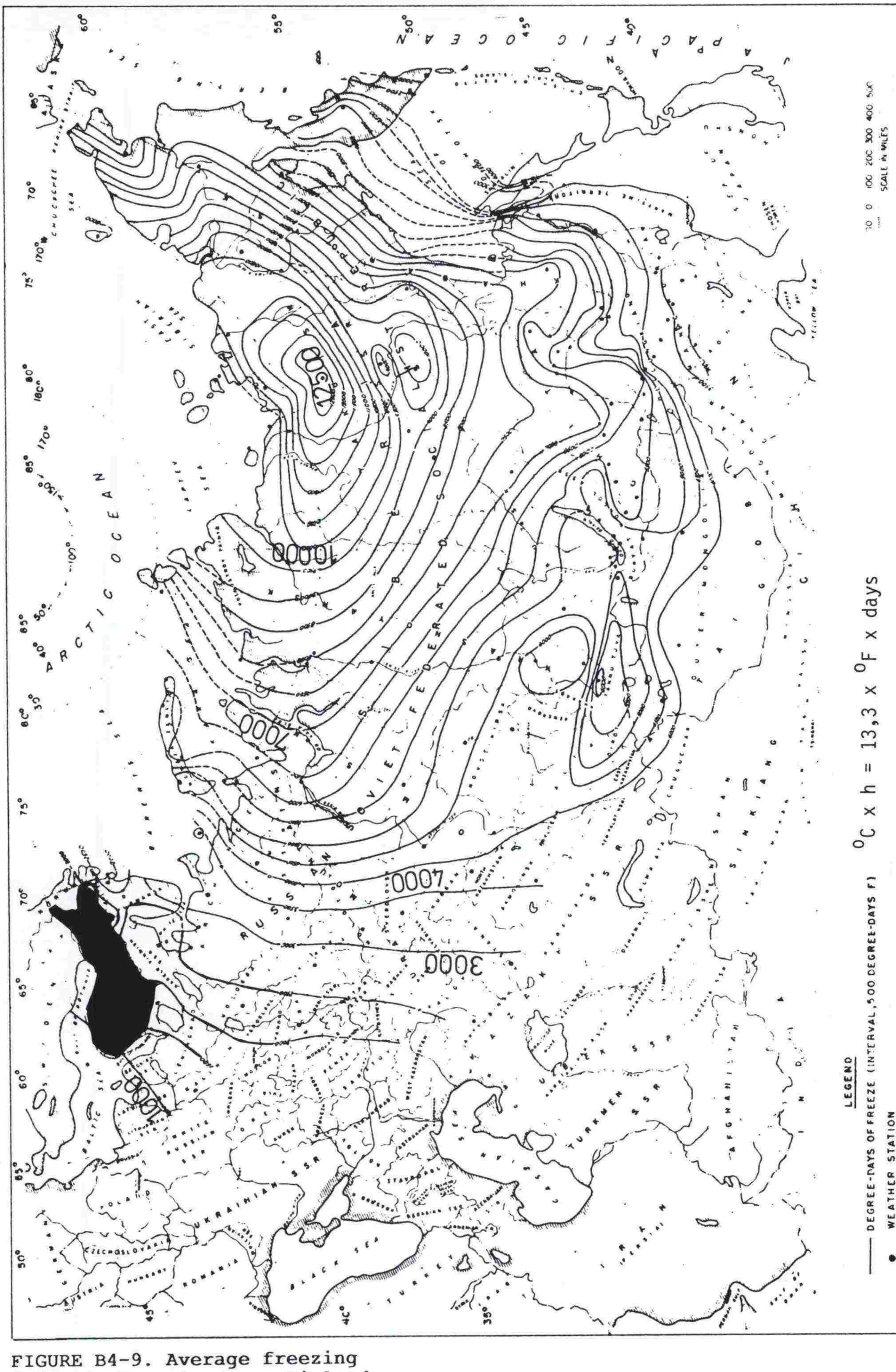


FIGURE B4-9. Average freezing index in Eurasia (e.g. Finland 10000-45000 °C x h, Eastern Siberia 100000-170000 °C x h) /3/

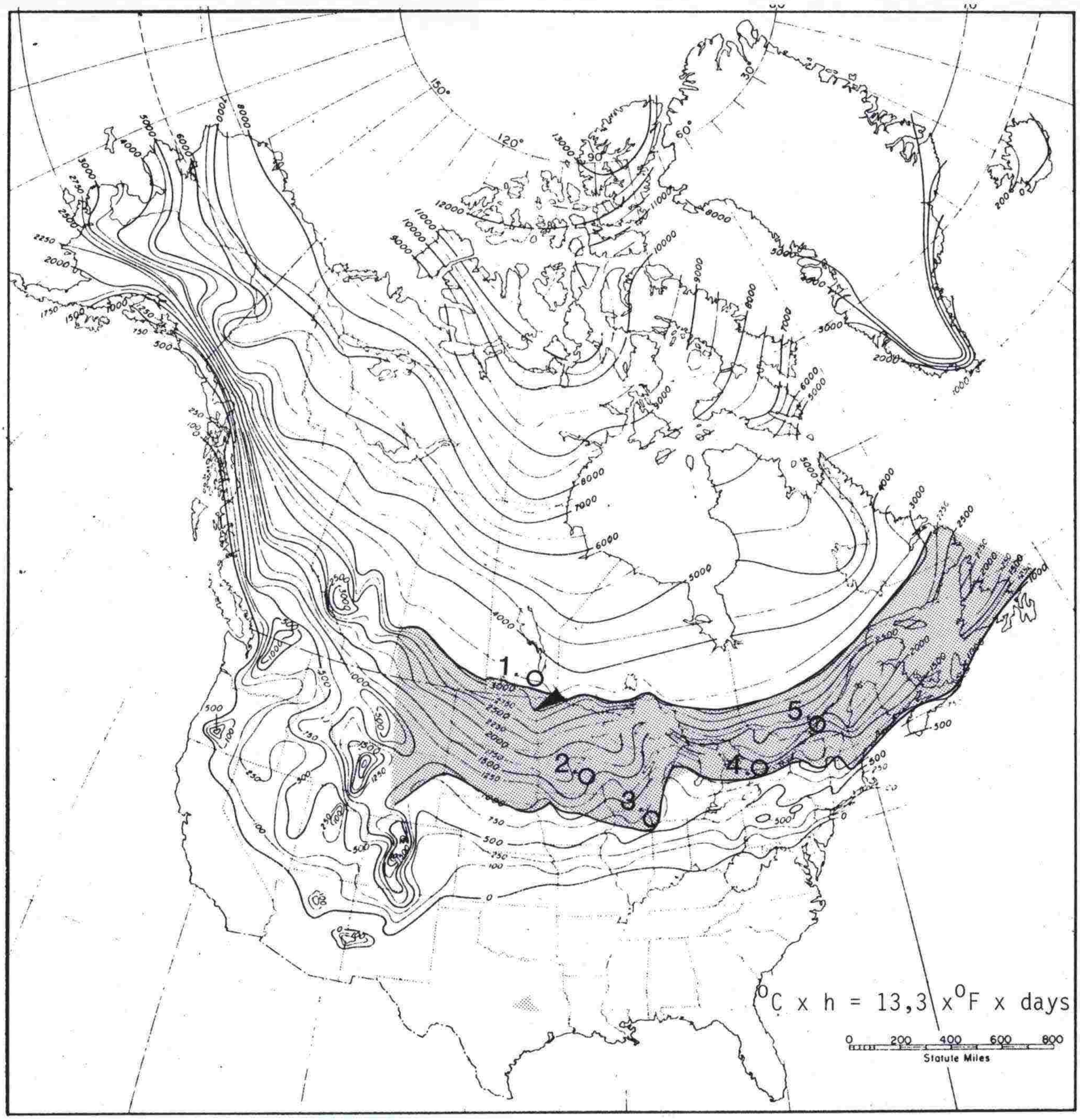


FIGURE B4-10. Average freezing index (°F x days) in North America /3/ (the shaded area corresponds to the Finnish conditions)

1. Winnipeg, Manitoba Can,
2. St. Paul, Minnesota
3. Chicago, Illinois
4. Toronto, Can,
5. Montreal, Can.

B 43 FROST ACTION DAMAGE ON ROADS AND STREETS

On the basis of the previous chapters it can be stated that frost action conditions are very unfavourable in Finland and thus frost action is a risk that must be taken into account in all road construction. However, the actual inconvenience of frost heave and a decreased spring bearing capacity depends greatly on local conditions and on the requirements set by the structure itself. Because evenness and a good load-bearing capacity are important to the function of roads and streets, frost-induced damage is most critical. Roads are always bare and free of snow which results in a maximum frost penetration. When running through the landscape roads always meet with variable soil and groundwater conditions which leads to considerable differences in frost heave and spring bearing capacity. These differences are evened out by designing the road structure properly, but in any case the durability and serviceability of the road are strained by the annual cycle of freezing and thawing.

B 431 Inconveniences of frost heave

Nonuniform frost heave, - abrupt changes in grade at the pavement surface, which can be tens of centimetres wide and essentially damage the evenness of the road in winter - is regarded as a primary inconvenience of frost action, Figure B4-11. The damage may be temporary, it is common that the evenness will be restored in the course of summer. When it is a question of greater heaves cracking of the pavement will always occur and the road will be exposed also to permanent damage. If frost action conditions are homogenous, frost heave can be considerably great, even as much as 10-20 cm without causing any great damage or significant unevenness. When total frost heave increases, also the risk for irregularities increases, Figure B4-12.

Frost heave causes also permanent damage. The annually repeated frost heave loosens the subgrade and the structural courses of the road during thawing and the recovery from frost heave won't be perfect. Permanent unevenness occurs and the service level of the road decreases in the course of years.

Design standards, frost action conditions and of course road categories determine which amount of frost heave or decrease in service level is regarded as detrimental in different countries. The margin values shown in Table B4-1 are recommended for different road categories in Finland. According to the recommendation total frost heave of more than 30 cm and short uneven bumps of 5-10 cm are considered detrimental on collector roads, whereas total frost heave of 4-7 cm and a single bump of 2-3 cm would be maximum values on motorways. At present a revision of these recommendation values is subject to consideration.

A reduction of the service level caused by differential frost heave in the course of years is regarded as a primary frost heave damage in the United States, Figure B4-13. A method to evaluate a decrease in the service level on the basis of the frost susceptibility of the subgrade, depth of frost penetration and drainage conditions is described in the newly published instructions of AASHTO, Figure B4-14 /15/.

FIGURE B4-11. Frost damage on Main Road 12, in the town of Hollola /III/ 1987, an asphalt pavement

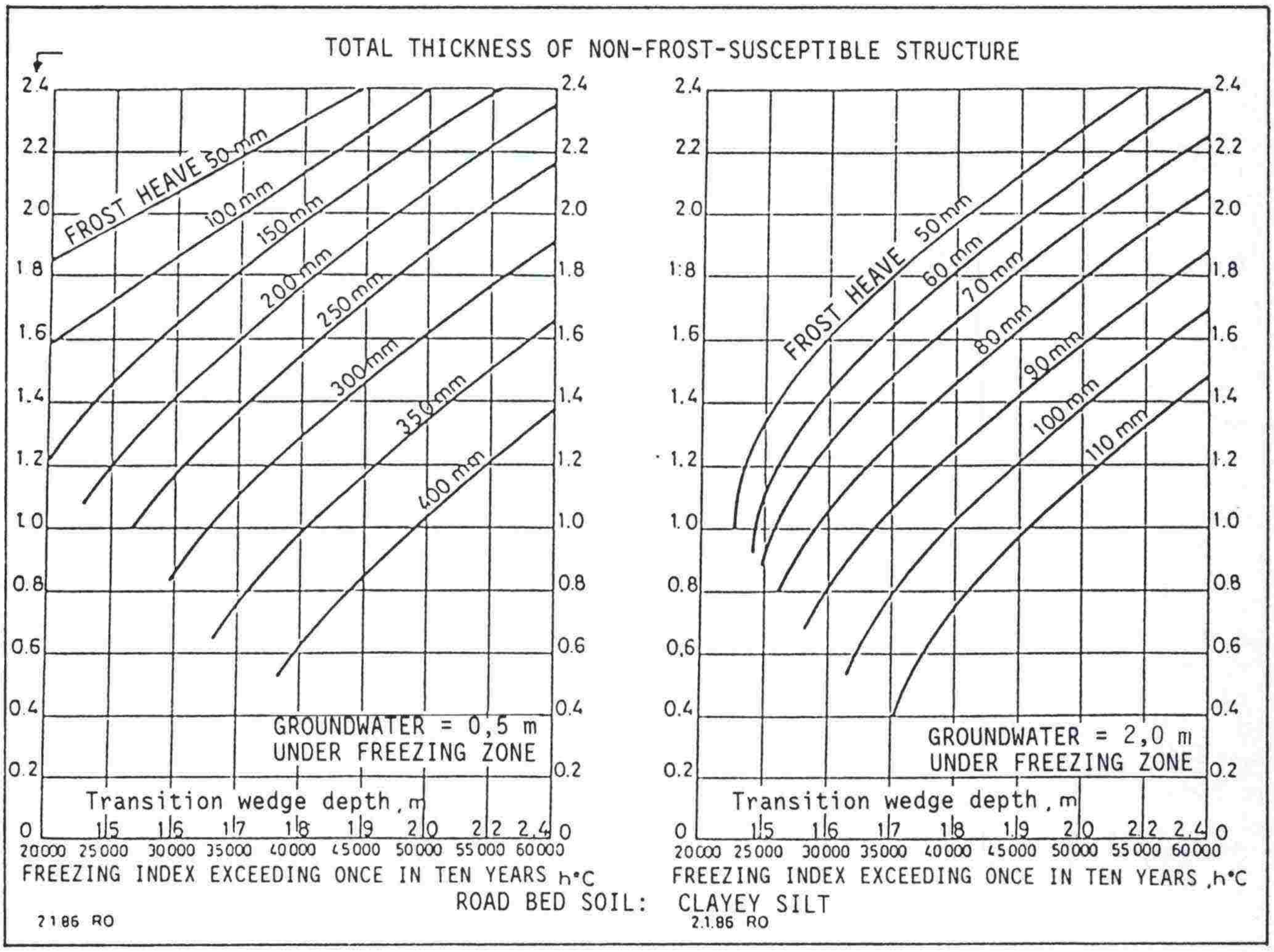


FIGURE B4-12. Frost heave as a function of freezing index and non-frost-susceptible structural thickness (According to R Orama)

TABLE B4-1. Recommended maximum values for changes in gradient and total frost heave (acc. to R Orama)

Functional class of the road	maximum change in gradient, o/oo	maximum total frost heave (mm)
Motorways	4...6	40...70
Main roads I and II	4...6	50...100
Regional roads	4...11	80...200
Collector roads	9...16	150...300
Connecting roads	15...	200...

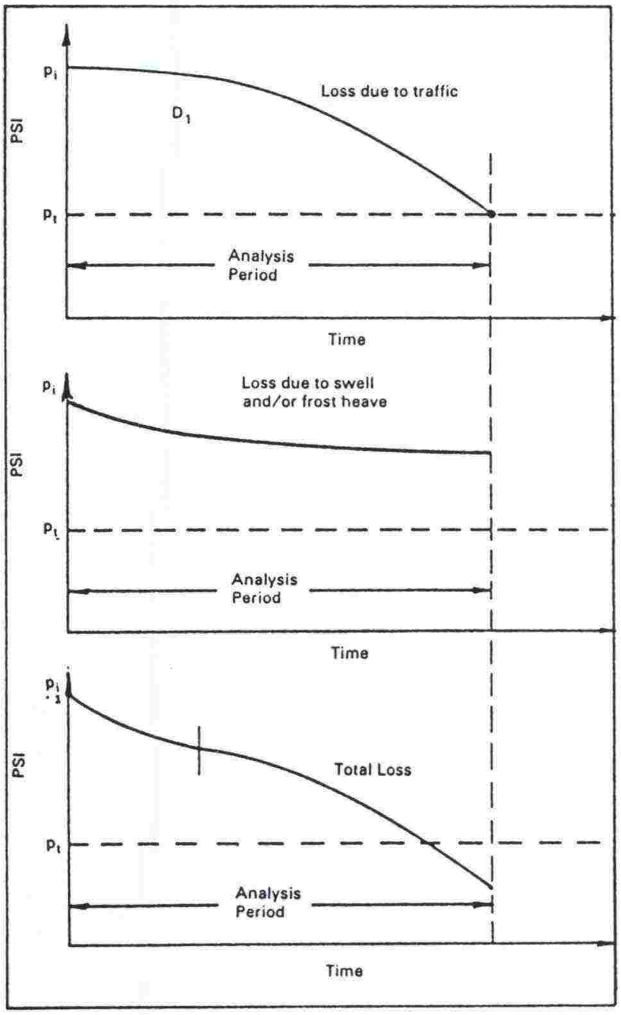


FIGURE B4-13. Decrease in the service level of the road as a time function according to American design instructions

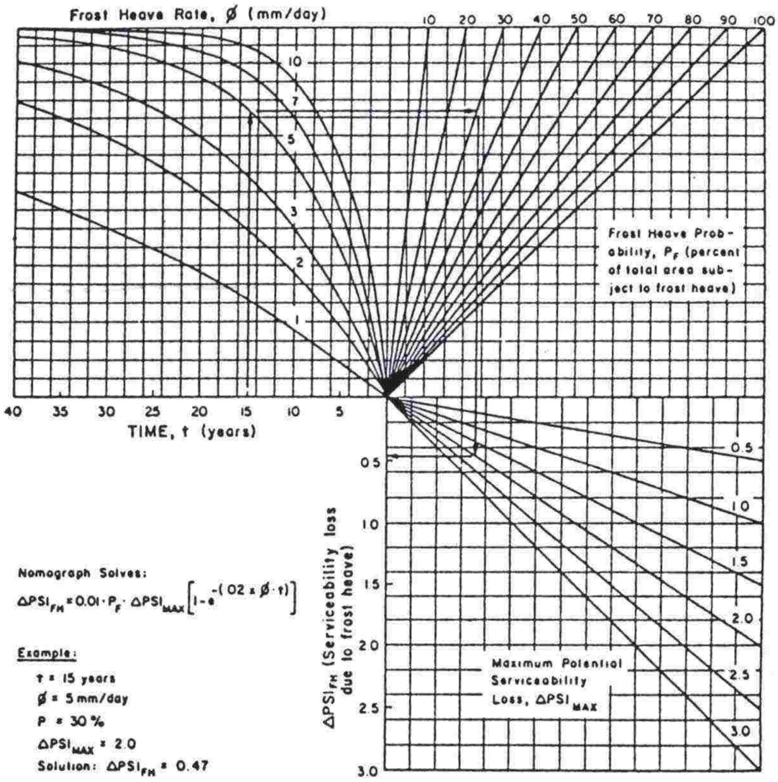


Chart for estimating serviceability loss due to frost heave.

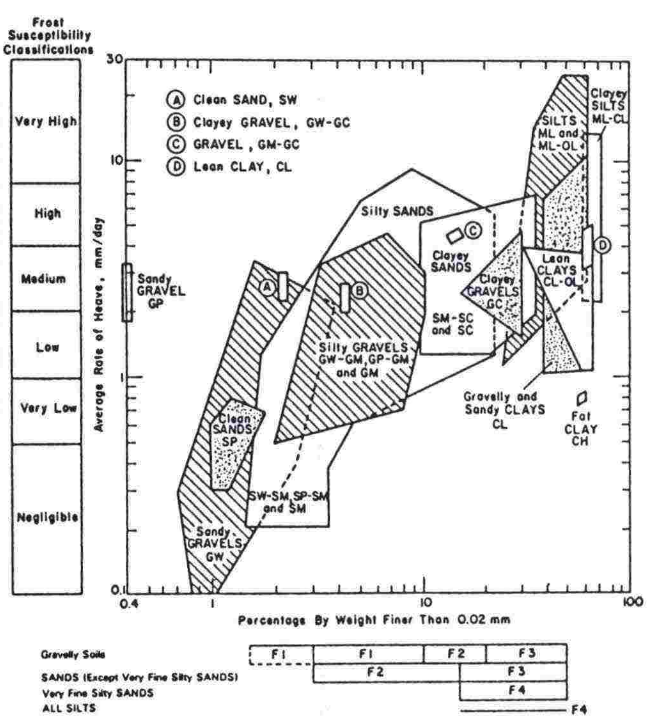
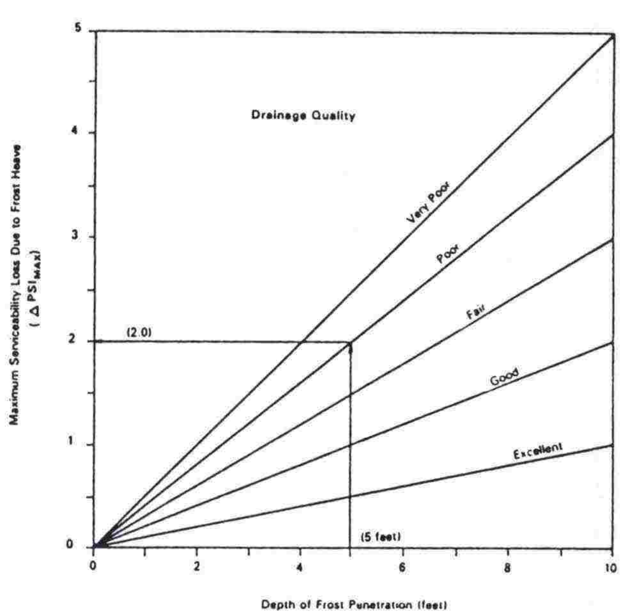


Chart for estimating frost heave rate for a roadbed soil, Part II (11).



Graph for estimating maximum serviceability loss due to frost heave.

FIGURE B4-14. Decrease in the service level due to frost heave according to American design instructions /15/

B 432 Inconveniences caused by a decreased spring bearing capacity

Variations in the load-bearing capacity of frost-susceptible soils (Figure B4-3d, e) during the seasons of the year are characteristic of seasonal frost action. The load-bearing capacity of frozen soil generally exceeds all the loading requirements, but significant loss of bearing capacity will take place during spring thawing periods followed by a recovery to the normal summer bearing capacity with the advancement of drying.

As far as the durability of the road structure is concerned this lowest value of the load-bearing capacity is decisive and that is why efforts are being made to measure the actual spring bearing capacities and to design the structures accordingly. Because it is difficult to reach the weakest load-bearing capacity with measurements, coefficients for load-bearing capacities of different seasons have been used to help in the design. The length of the seasons have been estimated on the basis of local temperature observations in some countries and thus a weighted bearing value has been obtained as a basis for the design. Finland and many other European countries use the weakest spring bearing capacity in their design, whereas the weighted bearing value is used among others in North America.

Although a decreased spring bearing capacity is primarily characteristic of fine-grained, frost-susceptible soils related to the melting of ice lenses (Figure B4-2), load-bearing capacity may decrease detrimentally also in non-frost-susceptible pavement structures. Especially in the beginning of the thawing period road edges are still frozen and normal drainageways overflow. Thus so much water can be dammed up on the pavement structure that the pore-water pressure under traffic loads will rise and the structure will loose some of its load-bearing capacity. This kind of a situation occurs usually only once a year in spring in Finland, but there are tens of mild periods during the winter in more southern seasonal frost countries. There the above phenomenon is the most essential disadvantage of frost action.

B 433 Pavement damage

Frost heaving reduces the regularity and service level of the road and the decreased spring bearing capacity tests its durability. Both inconveniences result in the danger of cracking and breaking the pavement. Typical frost action damage of asphalt pavements is longitudinal cracks of the roadway centerline, Figure B4-15. The pavement cracks at random at local frost bumps, Figure B4-11. Cracks may be serious and they may cause traffic restrictions and direct repairing need as shown in the figure. The decreased spring bearing capacity causes alligator cracks on an asphalt pavement and rutting at road edges.

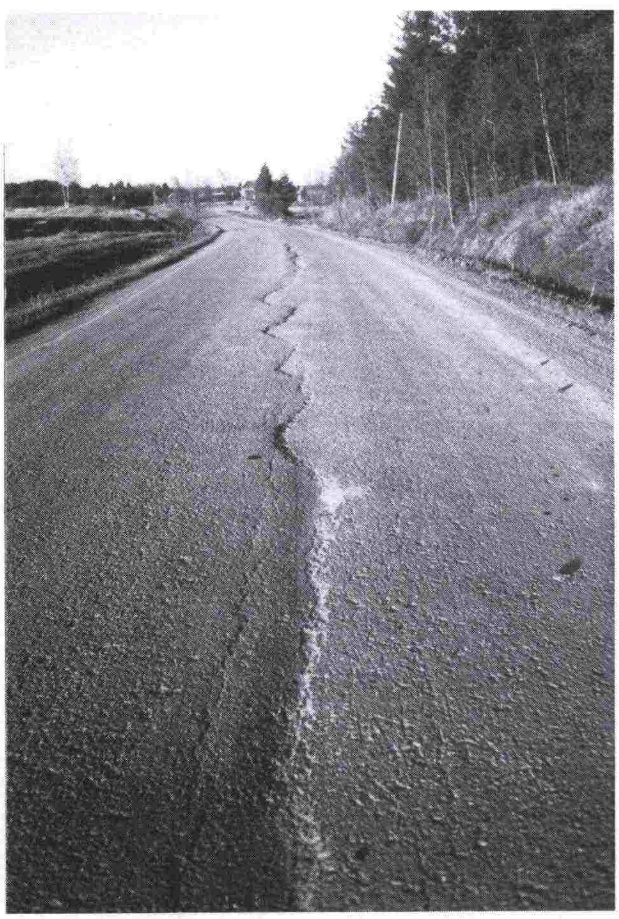


FIGURE B4-15. A longitudinal crack due to frost heave in the centre of road, an oil gravel pavement in southern Finland

On concrete pavements frost heaving results also in longitudinal cracks, Figure B4-16, or opening of longitudinal joints. A more typical frost damage, however, is faulting in transverse joints and at the edge of the pavement (Figure B4-17, B4-18), which may result from frost heave or from the decreased spring bearing capacity.

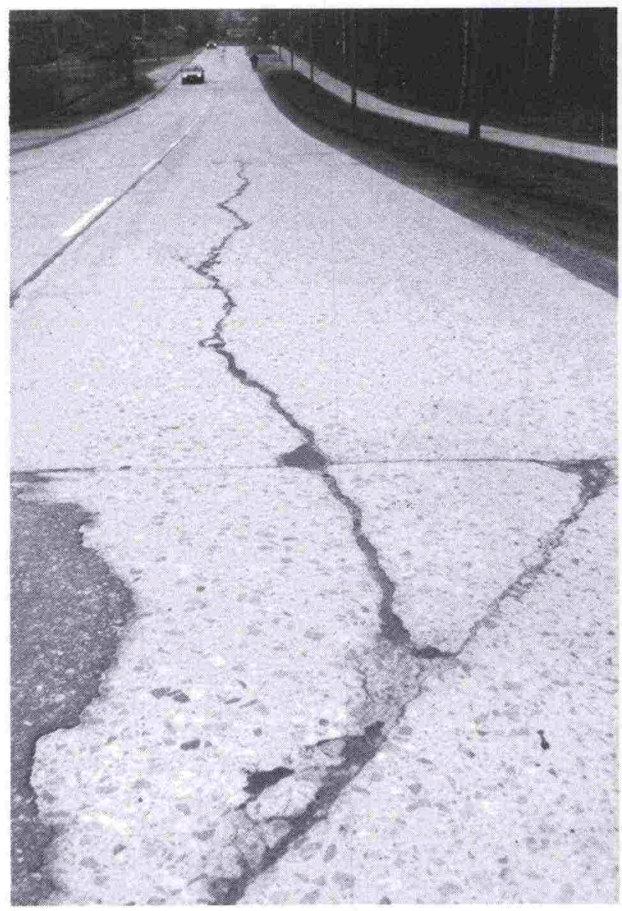


FIGURE B4-16. A longitudinal crack due to frost heave on a concrete pavement, Parainen

In the course of years this pavement damage produce deteriorating traffic damage or even destruction of the pavement itself when the cracking phenomenon advances. Cracks can be patched, pitched, reinforced or milled, but actual frost damage on the pavement cannot be removed by repairing the pavements; the repair must be undertaken by renewing the entire structure.

FIGURE B4-17. Faulting of slabs due to frost heave, Wisconsin USA (photo JR/1987)

B 434 Other frost action damage

Frost action causes migration of stones. Stones at the depth of frost penetration in the base structure are liable to migrate upward to the surface and cause unevenness in the course of time. Stones, the diameter of which is 15 cm, are already considered detrimental in a frost-susceptible subgrade in the United States, elsewhere stones of 30-60 cm are generally regarded as detrimental when thicker pavement structures are used. The annual freeze-thaw cycles cause also other than direct frost damage on the pavement structure. Both bound and unbound layers shrink during periods of frost, cracks will be formed on them and they appear as transverse cracks at regular distances on the surface. Frost shrinkage of concrete pavements is generally directed at joints and it won't cause damage.

FIGURE B4-18. Faulting between shoulder and pavement, Wisconsin USA

B 44 PREVENTION OF FROST DAMAGE

B 441 Methods to decrease damage

In seasonal frost areas prevention of frost damage dictates the boundary requirements for the structural design of the road. A good practice to avoid frost damage is to remove frost-susceptible soil and replace it with non-frost-susceptible material all the way down to the depth of frost penetration (deep road bed). This procedure is common in milder seasonal frost countries where the freezing index will remain below 3000 °C x h once in ten years and the frost depth is less than one metre.

Instead, in cold seasonal frost countries where frost action can penetrate as deep as to 2-3 m, only airfields and first-rate motorways are founded, when need be, to the non-frost-susceptible depth. A compromise is made elsewhere on the road network; frost heaving is allowed, the pavement structure and transition wedges even differential frost heave and differences in the spring bearing capacity, but they do not altogether prevent frost action in the subgrade. There are also other methods to reduce frost damage; these are mainly used to support the thickness design of the pavement structure. Such methods are among others the drawdown of the groundwater table, effective drainage of the pavements and thermal insulation. An estimate of the effectiveness of these measures in the prevention of frost damage is shown in Table B4-2.

B 442 Frost protection of the pavement in different countries

B 4421 Frost protection in Finland

According to the design instructions published by the Road and Waterways Administration in 1985 /16/ the structural courses of the road are first designed on the basis of the load-bearing demands of the road. The total thickness of the structure thus obtained shall be tested on the basis of frost protection demands. The starting point of the load-bearing design is the weakest spring bearing capacity and the E-modules of the available aggregates. The necessary layer thicknesses are defined by means of the Odemark formulae.

The intention of the frost protection is to restrict frost heaves to a certain level in each road category, Figure B4-1. The minimum requirements for a total thickness of the pavement structure, Figure B4-19, are based on long-term frost depth and frost heave measurements carried out by the Road and Waterways Administration in different parts of the country.

TABLE B4-2. Prevention of frost damage, efficiency of different measures

	Decrease in differential forst heave	Increase in spring bearing capacity
Non-frost-susceptible fill	xxx	xxx
Transition wedges	xxx	x
Homogenization	xx	x
Drainage of layers	x	xx
Rigider pavem. structure	x	xx
Drawdown of gwt	xx	xx
Thermal insulation	xxx	xxx

Alusrakenne

FROST PROTECTION

Thickness design

a) Vaatimustaso	b) Tien toiminnallinen luokka, [1] liikennemäärä ja päällystetyyppi [2]	c) Routaolosuhteet (kuva 32:1)	d) Eri pakkasmääriä (h °C) ja siirtymäkiilasyvyyksiä vastaavat rakenteen paksuudet [4) 5)] 20...25 000 1,5m	25...30 000 1,6m	30...35 000 1,7m	35...40 000 1,8...1,9m	40...45 000 1,9...2,0m	45...60 000 2,0...2,2m
I	· Moottoritiet, moottoriliikennetiet · Taajamien pääväylät, joissa on reunatuet ja sadevesiviemäröinti	Vaikeat [3)] Keskiv. Helpot	1,3...1,5m 1,0...1,3m 0,7...1,0m	1,4...1,6m 1,1...1,4m 0,8...1,1m	1,5...1,7m 1,2...1,5m 0,9...1,2m	1,6...1,9m 1,3...1,7m 1,0...1,4m	1,7...2,0m 1,4...1,8m 1,1...1,5m	1,8...2,2m 1,5...2,0m 1,2...1,7m
II	· Valta-, kanta-, ja seudulliset tiet, joiden KVL ≥ 1000 ajon/d · Taajamien muut yleiset tiet	Vaikeat [3)] Keskiv. Helpot	1,1...1,5m 0,7...1,0m —	1,2...1,6m 0,8...1,1m —	1,3...1,7m 0,9...1,2m —	1,4...1,9m 1,0...1,4m —	1,5...2,0m 1,1...1,5m —	1,6...2,2m 1,2...1,7m —
III	· Valta-, kanta-, ja seudulliset tiet, joiden KVL < 1000 ajon/d · Kokooja- ja yhdystiet, joihin tulee AB tai KAB	Vaikeat [3)] Keskiv. Helpot	1,1...1,5m 0,4...0,7m —	1,2...1,6m 0,5...0,8m —	1,3...1,7m 0,6...0,9m —	1,4...1,9m 0,7...1,1m —	1,5...2,0m 0,8...1,2m —	1,6...2,2m 0,9...1,4m —
IV	· Kokooja- ja yhdystiet, joihin tulee OS tai SOP	Vaikeat [3)] Keskiv. Helpot	0,9...1,5m — —	1,0...1,6m — —	1,1...1,7m — —	1,2...1,9m — —	1,3...2,0m — —	1,4...2,2m — —
V	· Kokooja- ja yhdystiet, joihin ei tule päällystettä eikä pintausta		— [6)]	— [6)]	— [6)]	— [6)]	— [6)]	— [6)]

a) category

b) I motorways, urban highways
II main roads, ADT > 1000
III main roads, ADT < 1000
IV collectors, oil gravel pavement
V collectors, gravel surface

c) frost conditions
V = severe
K = moderate
H = easy

d) thickness of pavement structure following frost index and wedge depth

REMARKS: mixed structure – category II
concrete pavement, –category I

TVL

IV 3.2-4 16.9.1985

Easy:

groundwater is far away (more than S+1 m) from the road surfice (most embankments, part of low cuts)

Medium:

groundwater is near (not more than S+1 m) the road surfice, but the base stucture is homogeneous (most cuts, part of low embankments)

Difficult 1:

groundwater is near (not more than S+1 m) the road surface and the base structure is unhomogeneous: there are boulders (>0,5 m), different kinds of layers as to permeability, rock above the transition wedge depth (short sections in cuts or on low embankments

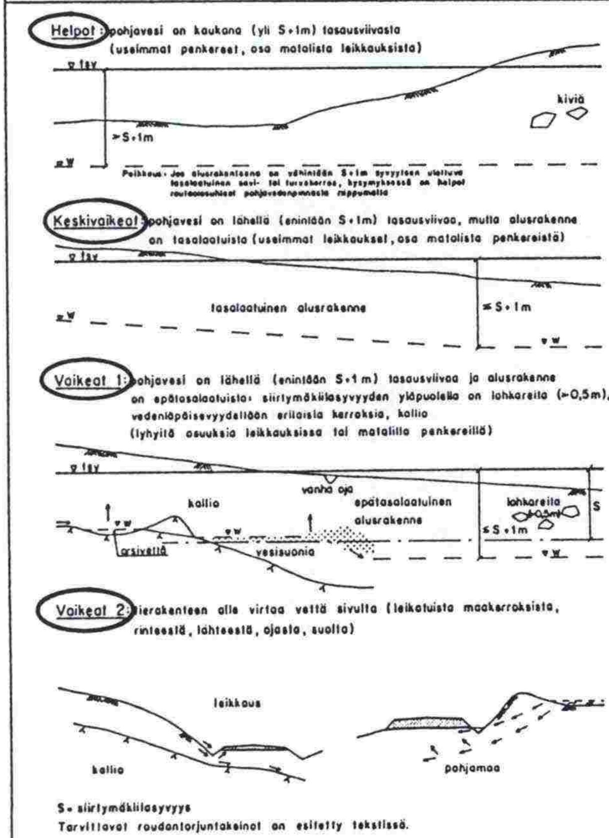
Difficult 2:

water from sides flows under the road structure (from cut soil courses, slopes, spring, ditch, swamp)

Alusrakenne TVL 16.9.1985

FROST PROTECTION

Frost conditions of base structure



IV 3.2-3

FIGURE B4-19. Frost protection of the road structure in Finland /16/

The design instructions of the Road and Waterways Administration, Figure B4-19, stipulate that concrete-paved roads are to be built according to standard 1, in other words, a thicker pavement structure should be used when building concrete pavements on inferior roads and streets than when building asphalt pavements. This view is based on the opinion that a concrete pavement does not withstand the frost heaving allowed on inferior roads according to Table B4-1 as well as an asphalt pavement does.

The load-bearing design of concrete pavements according to the Finnish design instructions is undertaken in the same way as on asphalt-paved roads, the load-bearing target being, however, 125 MN/m^2 for concrete pavement base.

A development project for structural design of city streets has been started by Suomen kaupunkiliiitto in the mid-1980s. In this connection a new frost design method has been made out in the geotechnical laboratory of VTT, Figure B4-20. The method introduces certain new practices for the frost design: more design frost indices than before and a clearer use of the allowed frost heave as a design element. The method is unofficial so far. It does not commit itself as regards concrete pavements, /24/.

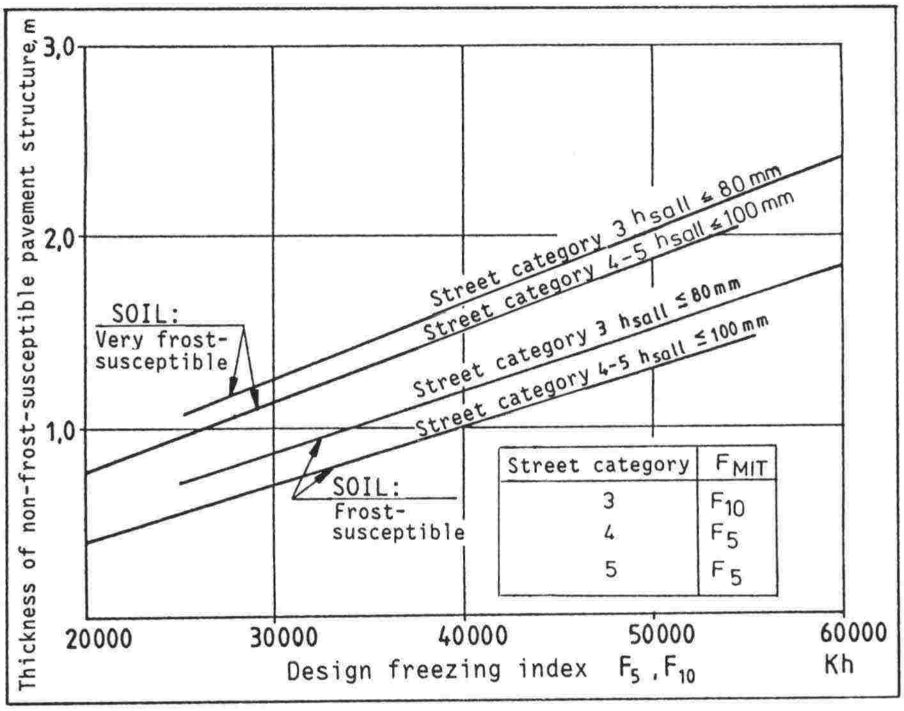


FIGURE B4-20. Suggestion for frost protection of a street structure/24/

B 4422 The road structure design in the United States and Canada

In North America the design methods of the pavement structure are still in one way or other based on the road test made by AASHO in the 1960s. The design methods of different states often differ from each other and there are modifications of the original instructions completed by own experiences. Mechanistic methods already well-known in Europe- based on theoretical calculations and on utilization of different material qualifications - have not yet gained ground to the same extent in North America as in Europe. The design instructions published by the research laboratory of cold areas CRRL are usually carried out in the pavement structure design in cold seasonal frost states. Two parallel design methods of this recently revised instruction will be studied in the following, /3, 1, 2/.

a) Design on the basis of frost penetration

The aim of the method is to restrict frost heaving to the level of about 10 cm by replacing 80 % of the depth of frost penetration with non-frost-susceptible pavement structure. The frost penetration depth in non-frost-susceptible soils is defined as a function of the freezing index, the volume weight of the material and the moisture content, Figure B4-21. The necessary pavement structure thickness is then defined as a function of the water content of the subgrade and pavement structure, Figure B4-22. No significance is given to bound layers in the frost protection of this method, their thickness is separately defined and it is added to the pavement structure thickness obtained in the frost protection design. The method does not separate asphalt and concrete pavements, pavement structure thicknesses are the same for both of them.

b) Design for reduced spring bearing capacity

This method entirely ignores the reduction of frost heaving. A thorough homogenizing of the base structure and compaction to a depth corresponding to 1/2 - 1/3 of the frost penetration depth in Figure B4-21 is effected to avoid frost damage. Transition wedges are installed if necessary. Unbound layers and an asphalt pavement are dimensioned to this homogenized subgrade layer by layer. Measured spring bearing capacity values are not used; an average value defined on the basis of the soil type is used for load-bearing design, it corresponds to the weighted average of the load-bearing differences during the seasons. As far as asphalt pavements are concerned this method leads to pavement structure thicknesses in the size range of 60 cm.

A concrete pavement can be dimensioned to a homogenized base structure using a sand layer of only 10 cm as a base. The weighted annual average defined on the basis of the soil type is used as a modulus of subgrade reaction (k). A decrease in the service level, PSI, allowed to the pavement during the service life together with traffic loads and the modulus of subgrade reaction are used as variables in the slab thickness design. If the design of the slab aims at an uniform and a thin slab (= 20 cm), thicker gravel layers or a bound base course under the slab are used to raise the modulus of subgrade reaction. In practice the slab thicknesses vary between 20 and 28 cm.

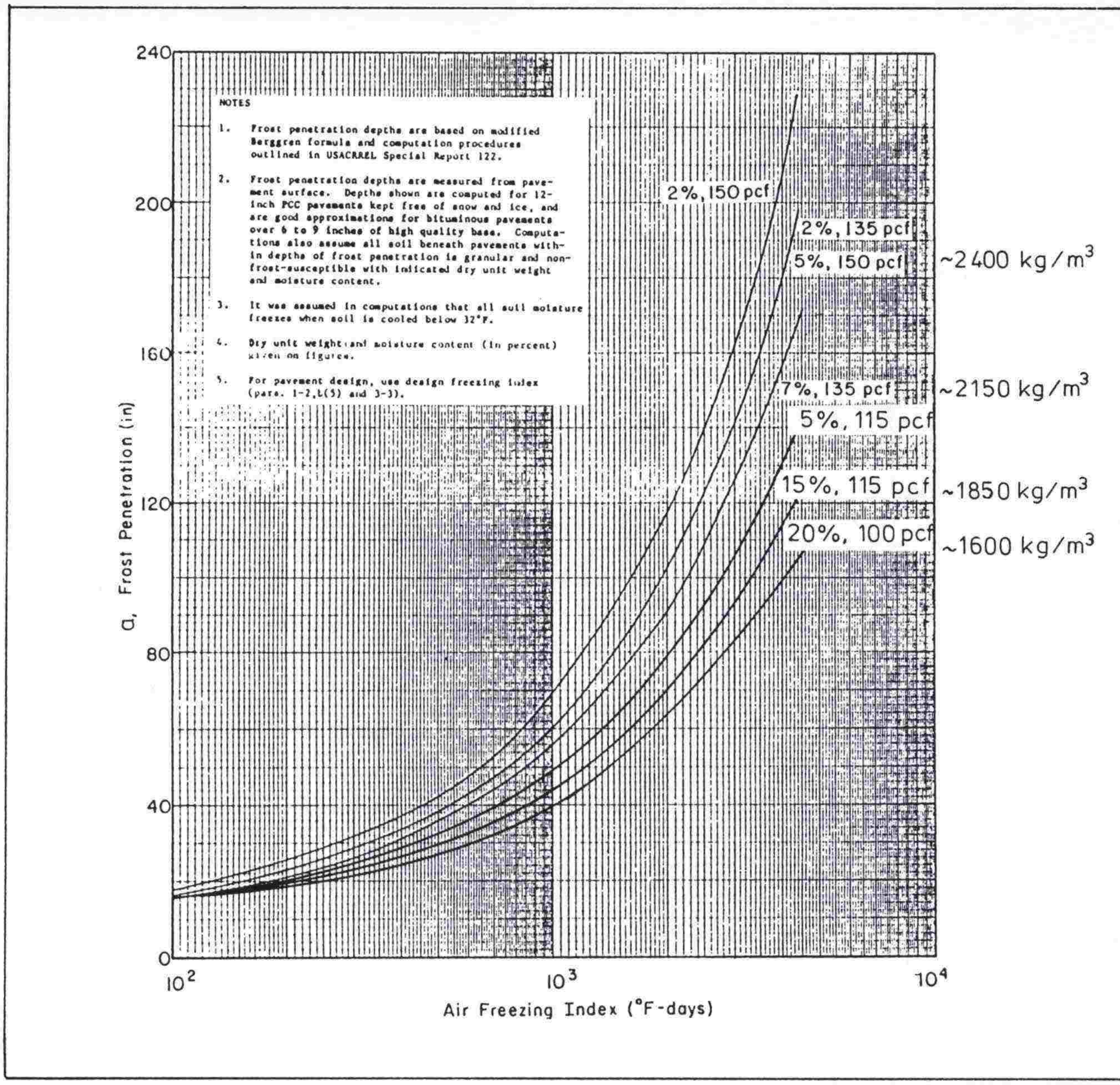
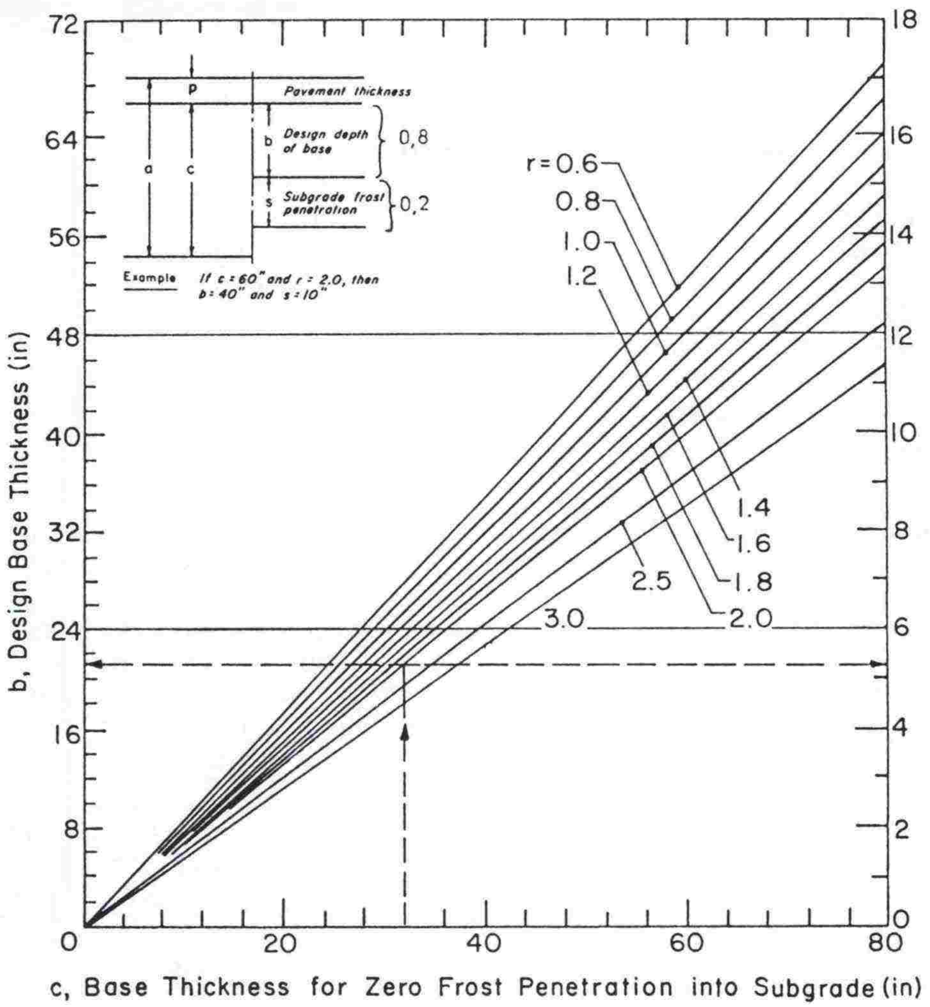


FIGURE B4-21. Definition of frost penetration depth, the CRRL method

c) Scopes of the alternatives

Alternative b) should be the major design method according to the CRREL instructions and this has also been the practice in USA as noticed during the research journeys. This design method makes use of the load-bearing advantage of the concrete pavement also in frost-susceptible circumstances. The total thickness of the unbound slab structures and the pavement is only about half of that of an asphalt-paved structure. The choice between asphalt and concrete is made by comparing construction costs.

However, method b) is not suitable if base conditions are very variable or exceptional requirements are set for the evenness. Runways on airfields are generally dimensioned according to alternative a). If the total thickness of the pavement structure would rise above 150 cm, the structure will be restricted to this



NOTES

$\underline{a}$ = Combined thickness of pavement and non-frost-susceptible base for zero frost penetration into subgrade.

$\underline{c} = \underline{a} - \underline{p}$

$\underline{w}_b$ = Water content of base.

$\underline{w}_s$ = Water content of subgrade.

$\underline{r} = \frac{w_s}{w_b}$ Not to exceed 2.0 for type A and B areas on airfields and 3.0 for the other pavements.

FIGURE B4-22. Definition of pavement structure thickness on the basis of frost penetration depth; the CRRL method /3/

thickness and other measures for further assurance are taken. As far as concrete pavements are concerned slab lengths will be shortened or reinforcement will be used. In asphalt-paved structures also thermal insulation may be used to keep the structural thickness reasonable.

The design in the Finnish terrain and soil circumstances shall to a great extent be carried out according to method a) when the CRREL methods are applied. However, in method b) there would be much to learn about the homogenization of subgrade and removal of stones as a levelling method of differential frost heave.

B 4423 Frost design of the road structure in Switzerland

Frost circumstances in Switzerland are very similar to those in Southern Sweden. The highest design freezing indices are in correlation to the lowest Finnish indices (about 20000°C x h). The frost design of the road structure is based on own experiences and on the AASHO road test /4/. Frost conditions are defined as easy (Figure B4-23) when the depth of frost penetration is not more than 1,4 m; frost protection is actually studied only when the depth of frost penetration is more than 1,4 m and the groundwater table is high. The total thickness of the pavement structure is dimensioned so that 60 % of the frost penetration depth will be compensated by non-frost-susceptible material, Figure B4-24. The thickness of the structure for an asphalt-paved road in severe conditions can be obtained from the diagramme in Figure B4-25. 10 % of the thickness can be reduced in less severe conditions. As to concrete pavements 15 % of the thickness is reduced. According to the Swiss standards this reduction is justified, because it is not necessary to take into account the decrease in the spring bearing capacity. To control differential frost heave, homogenizing frost-susceptible subgrade soil and transition wedges down into the non-frost-susceptible depth are emphasized in the standards. The spring bearing capacity is taken into account

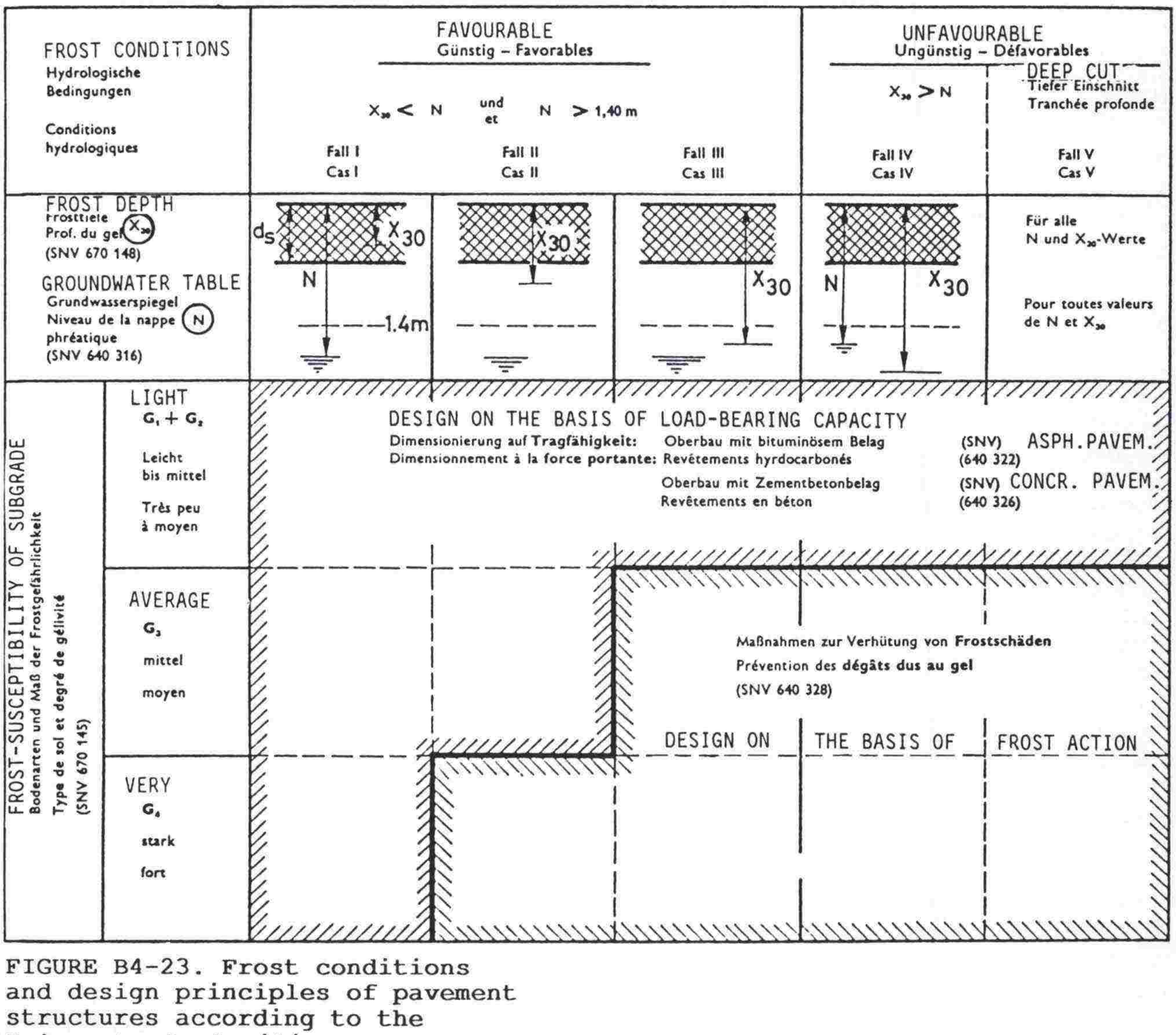


FIGURE B4-23. Frost conditions and design principles of pavement structures according to the Swiss standards /4/

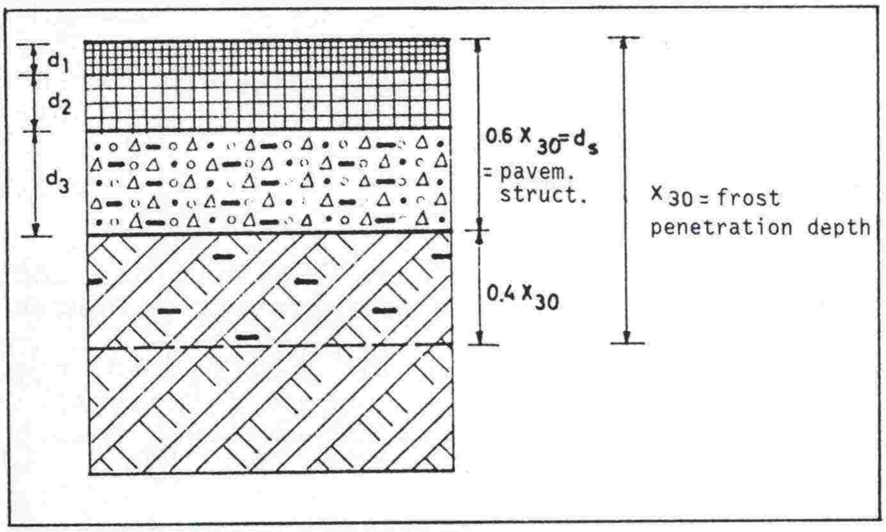


FIGURE B-24. Frost protection principle according to the Swiss standards /4/

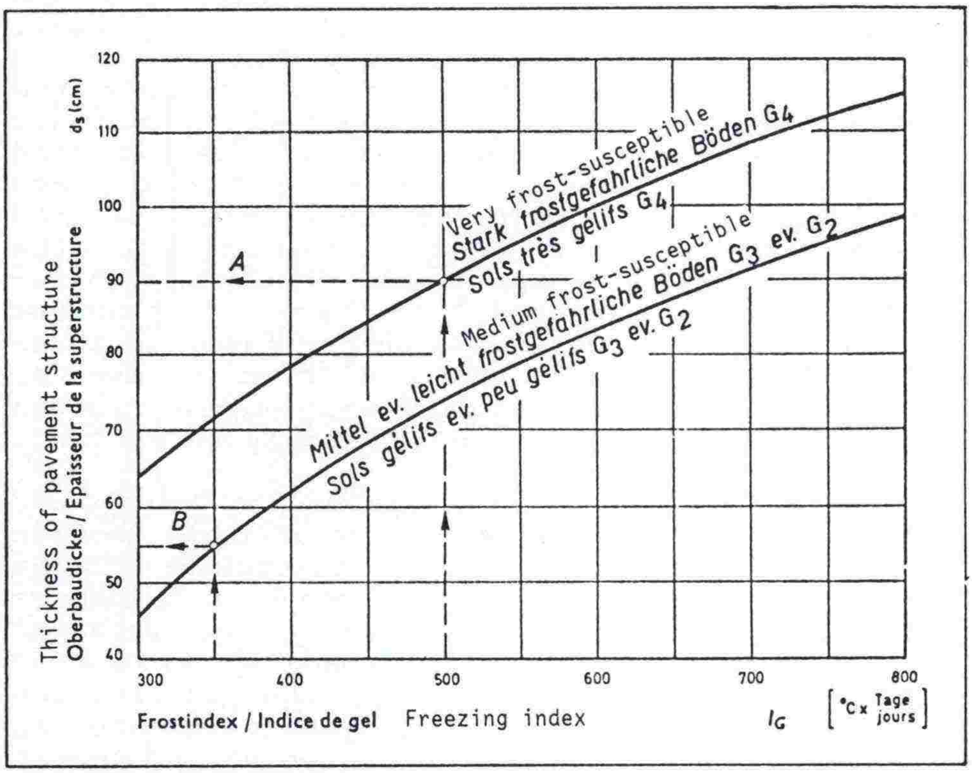


FIGURE B-25. Total thickness of the pavement structure according to the Swiss standards /4/

in the load-bearing design so that the structure dimensioned to the average load-bearing capacity of the subgrade soil and to the traffic of the entire year is tested by means of the load-bearing capacity and traffic during thawing. The decrease in the spring bearing capacity is ignored in the design of concrete pavements.

B 4424 Frost design of road structures in Norway

Freezing indices are the most important variables in the frost design principles /10/ of the Norwegian design standards, Figure B4-26. The design freezing indices and the corresponding structural thicknesses are defined beforehand. Local frost conditions and traffic flows determine whether the dimensioning is carried out as a load-bearing or as a frost protection design. Also variable freezing index probabilities are used in the design in Norway. The highest freezing index occurring only once in ten years is used only for greater traffic flows in difficult traffic conditions, otherwise the design is based on freezing indices occurring every five or every two years. Maximum values have been defined to the total thickness of the pavement (1.2, 1.5 and 1.8 m); these values are not exceeded although a local freezing index would so imply. The frost protection of concrete pavements leads to the same pavement structure thicknesses (Figure B4-26) as with asphalt pavements - only with the following exception. A frost protection thickness of not more than 1,2 m is required for concrete pavements under traffic flows of 1000-10000 vehicles/day in medium difficult conditions while asphalt pavement is designed just on the basis of the load-bearing capacity in such conditions. On the other hand, concrete-paved roads under the same traffic flow in easy frost conditions are designed only on the basis of the load-bearing capacity which leads clearly to a thinner pavement structure than on asphalt-paved roads.

B 4425 Frost design of the road structure in Sweden

A new design method of the pavement structure for asphalt-paved roads /8/ has been developed in Sweden in 1984. The method is based on the results and analytical calculations obtained from field and laboratory tests. The design is carried out by means of tables, Figure B4-27. Alternative structures can be chosen when the traffic flow, quality of the subgrade soil, freezing index and the drainage conditions are known. In the frost protection design the country is divided only into three areas. Total thicknesses of pavement structures according to subgrade soils and drainage conditions have been defined for them. According to the design tables the maximum thicknesses of the pavement structure depending on the freezing index are 1000 - 1100 mm in Sweden. These are clearly smaller figures than those in Norway and Finland.

Concrete pavements are not included in the official Swedish road design instructions. The same frost protection thickness as in the official instructions of asphalt roads /9/ has been mentioned in the instruction draft of concrete pavements. In the manual "Betong på mark" /29/ of 1985 a design principle for concrete pavements has been described according to which the total thickness of the pavement structure would remain 600 - 700 mm at the most, i.e. about 35 % smaller than that of asphalt-paved roads, Figure B4-28.

	Grunnforhold CONDITIONS	Årsdøgntrafikk ÅDT	ASPH.PAVEM. Veg med bitum.dekke: steinmat.	bark	isolasjon	CONCR.PAVEM. Veg med betongdekke: steinmat.	isolasjon	Merknad
DIFFICULT	A Sterkt varierende grunnforhold. Store, ujevne telehivinger er ventet	under 1000	B	B	B	-	-	h_2 etc. er forklart på s. 36
		1000–10000	h_5, max 1,5 m	h_5	h_{10}	h_5, max 1,5 m	h_{10}	
		over 10000	h_{10}, max 1,8 m	h_{10}	h_{10}	h_{10}, max 1,8 m	h_{10}	
AVERAGE	B Noe varierende grunnforhold. Endel ujevne telehivinger er ventet	under 1000	B	B	B	-	-	Verdier for h_2, h_5 og h_{10} er gitt i fig. 24, s. 61
		1000–10000	B	B	B	h_2, max 1,2 m	h_{10}	
		over 10000	h_5, max 1,5 m	h_5	h_{10}	h_5, max 1,5 m	h_{10}	
EASY	C Forholdsvis homogene grunnforold. Bare små, ujevne telehivinger er ventet	under 1000	B	B	B	-	-	B angir at bæreevnemessig dim. ansees tilfredsstillende
		1000–10000	B	B	B	B	B	
		over 10000	h_2, max 1,2 m	h_2	h_{10}	h_2, max 1,2 m	h_{10}	

a) Frost design principle in different conditions

B means that load-bearing capacity design is sufficient

Freezing index	Probability to exeed the value in the design year	Thickness of non-frost-susceptible layer
F_2	50 %	h_2
F_5	20 %	h_5
F_{10}	10 %	h_{10}
F_{100}	1 %	h_{100}

b) Design freezing indices and corresponding structural thicknesses

KOMMUNE Fylkesvis ordnet med offisiell nummerering Areurtall 1975	SAND/GRUS h_2	h_5	h_{10}	h_{100}	BARK h_2	h_5	h_{10}	h_{100}	ISOLASJON h_2	h_{10}	h_{100}
01 ØSTFOLD											
0101 Halden	105	150	180	200	20	36	46	53	2,0	3,5	4,0
0102 Sarpsborg	105	150	180	200	20	36	45	53	2,0	3,5	4,0
0103 Fredrikstad	90	135	165	210	16	31	41	57	1,5	3,5	4,5
0104 Moss	100	140	170	195	18	34	43	50	1,5	3,5	4,0
0111 Hvaler	80	105	135	160	11	20	31	39	1,0	2,5	3,5
0113 Borge	90	135	165	210	16	31	41	57	1,5	3,5	4,5
0114 Varteig	105	150	180	200	20	36	45	53	2,0	3,5	4,0
0115 Skjeberg	105	160	180	200	20	36	46	53	2,0	3,5	4,0
0118 Aremark	125	165	195	215	28	41	50	58	2,5	4,0	4,5
0119 Marker	140	180	205	230	34	45	55	63	3,0	4,5	5,0
0121 Rømskog	155	190	215	240	38	48	59	67	2,0	4,5	5,5
0122 Trøgstad	125	165	195	220	28	41	50	60	2,5	4,0	5,0
0123 Spydeberg	125	165	195	220	28	41	50	60	2,5	4,0	5,0
0124 Askim	125	165	195	220	28	41	50	60	2,5	4,0	5,0
0125 Eidsberg	125	165	195	220	28	41	50	60	2,5	4,0	5,0

c) Pavement structure thicknesses in various communes

FIGURE B4-26. Frost protection principles of the road structure according to the Norwegian standards /10/

Trafikklass		5	6	7
Dimensionerande trafik		500–1 500	1 500–3 000	> 3 000

Lager		Ungefärlig tjocklek, mm		
Slitlager	Y1B Y1G Y2B Y2G OG			
	MAB,HAB,TOP	35	35	35
Bärlager AG		95	145	195

Lager		Minsta tjocklek, mm		
Slitlager + Bärlager AG		130	180	230
Slitlager + bärlager, totalt	På mtrl F	200	250	250
	På övriga mtrl	250	300	350

Material		Får inte ligga närmare vägytan än, mm								
F A B G		200 250 350 400			250 300 400 500			250 350 450 500		
		Medelköldmängd, d°C								
		< 200	200–1 000	> 1 000	< 200	200–1 000	> 1 000	< 200	200–1 000	> 1 000
C, DS	vd väldränerad	400	400	500	500	500	600	600	600	700
	nd normal - " -	500	600	700	600	700	800	700	800	900
D1, ES	vd	600	600	700	700	700	800	800	800	900
	nd	700	800	900	800	900	1 000	900	1 000	1 100
D2	vd	700	800	900	800	900	1 000	900	1 000	1 100
	nd	800	800	900	900	900	1 000	1 000	1 000	1 100
E		1 000			1 100			1 200		

Fortsättning Tabell 1:06–4 Dimensionering av grusbitumenöverbyggnad, GBÖ

FIGURE B4-27. An example of the pavement structure design table according to the Swedish standards /8/

Tabell 1.4 *Dimensionering av betongöverbyggnad BtgÖ vid olika underlag och trafikmängd.*

Materialgrupp	Terrassens dräneringsgrad[a]	Medelköldmängd °C · d	Materialets minsta avstånd från färdig yta (mm) Trafikklass 0	1	2 Antal tunga fordon 2	10	50	3	4	5
A	–	–	80	90	130	140	150	160	170	180
B	–	–	80	90	130	140	150	160	170	180
C	v	0–1000	80	90	300	300	300	300	300	350
	v	>1000	80	90	300	300	300	300	300	350
	n	0–200	80	90	300	300	300	300	300	350
	n	200–1000	80	90	300	300	300	300	300	350
	n	>1000	80	200	300	300	300	300	350	450
	o	0–200	80	150	300	300	300	300	300	350
	o	200–1000	80	250	300	300	300	300	350	450
	o	>1000	80	300	400	400	400	400	450	500
D1	v	0–1000	180	200	300	300	300	300	350	400
	v	>1000	180	250	300	300	300	300	350	450
	n	0–200	180	200	300	300	300	300	350	400
	n	200–1000	180	250	300	300	300	350	400	450
	n	>1000	180	300	300	300	300	400	450	500
	o	0–200	180	250	300	300	300	300	400	450
	o	200–1000	180	300	350	350	350	450	500	550
	o	>1000	180	350	400	400	400	500	550	600
D2	v	0–200	230	250	300	300	300	350	400	450
	v	200–1000	230	300	350	350	350	400	450	500
	v	>1000	230	350	400	400	400	450	500	550
	n	0–200	230	300	350	350	350	400	450	500
	n	200–1000	230	300	350	350	350	400	450	550
	n	>1000	230	350	400	400	400	450	500	600
	o	0–200	230	300	350	350	350	400	450	550
	o	200–1000	230	400	450	450	450	500	550	650
	o	>1000	230	450	500	500	500	550	600	700
E1	–	–	230	400	450	450	450	450	500	600
Betongbeläggning min tjocklek (mm)[b]			80	90	130 (140)	140 (150)	150 (160)	160 (180)	170 (210)	180 (220)

[a] v = väldränerad, n = normaldränerad, o = odränerad
[b] Angivna värden avser kontinuerligt rullande trafik. Värdena inom parentes avser trafik, där de tunga fordonen ofta stoppar upp (t ex gatukorsningar med signalreglering eller stopplikt).

v = väldränerad
n = normal - " -
o = odränerad

FIGURE B4-28. Design of the concrete-paved structure according to the Swedish instructions 'Betong på mark' /29/

TABLE B4-3. Frost design principles in different countries

	Design frost index (°C x h)	Design principle	Differences in concrete and asphalt structures in frost design
Southern seasonal frost action countries	F_{10} < 3000	100 % (of the frost penetration depth is compensated by non-frost-susceptible material, Fig. B4-23)	NO DIFFERENCES
USA and Canada	F_{10} < 60000	80 % (Fig. B4-22) or homogenization + load-bearing design based on the annual weighted bearing value	NO DIFFERENCES (homogenization leads to a thinner structure when concrete)
Switzerland	F_{10} < 20000	60 % (Fig. B4-24) the base structure can also be homogenized and stabilized	15 % THINNER WHEN USING CONCRETE
Norway	F_{10} 3000...55000	Standard structural thicknesses defined according to the traffic flow, frost conditions and frost index (Fig. B4-26)	GENERALLY NO DIFFERENCES
Sweden	F_{10} 3000...55000	Standard structural thicknesses defined according to the traffic flow, frost conditions and frost index (Fig. B4-27)	A STRUCTURE WHICH IS 35 % THINNER THAN THAT OF ASPHALT-PAVED ROADS SUGGESTED FOR CONCRETE ROADS
Finland	F_{10} 20000...60000	Minimum thicknesses and variation limits according to the road class, drainage conditions and frost index (Fig. B4-19)	NO DIFFERENCES in the motorway class. On all inferior roads and streets a thicker structure is required.

B 4426 Summary of frost design differences

The following differences can be seen in the frost protection of concrete and asphalt pavements based on the above descriptions, Table B4-3.

Generally there are no considerable differences in the frost protection of rigid and flexible pavement structures. If there are differences they usually suggest a thinner structure for concrete pavements. Only the Finnish and partly the Norwegian design instructions refer to the opinion that a stricter attitude towards frost heave should be taken when designing concrete pavements. On the other hand, the above description reveals how different countries make out their design instructions and collect their experiences in very variable circumstances.

As far as the entire country is concerned Finland has apparently the most difficult frost action circumstances of the seasonal frost countries studied, but Finland takes also the smallest risks in the design of the frost protection of new roads.

B 443 Other means to prevent frost damage

In addition to the non-frost-susceptible pavement structure and to homogonizing the subgrade soil also many other measures can be taken to prevent frost damage, Table 2.

a) Transition wedges complete the frost protection of the pavement structure everywhere where the thickness of the pavement structure is less than frost penetration depth. The significance of the transition wedges is emphasized in the Scandinavian design and construction practice, the Finnish practice in Figure B4-29. In some cases in Switzerland transition wedges can be ignored or frost penetration made lower if full-depth cement-stabilized pavement structures are used. Elsewhere the surface slab or any other bound structure has generally no influence in the need of transition wedges.

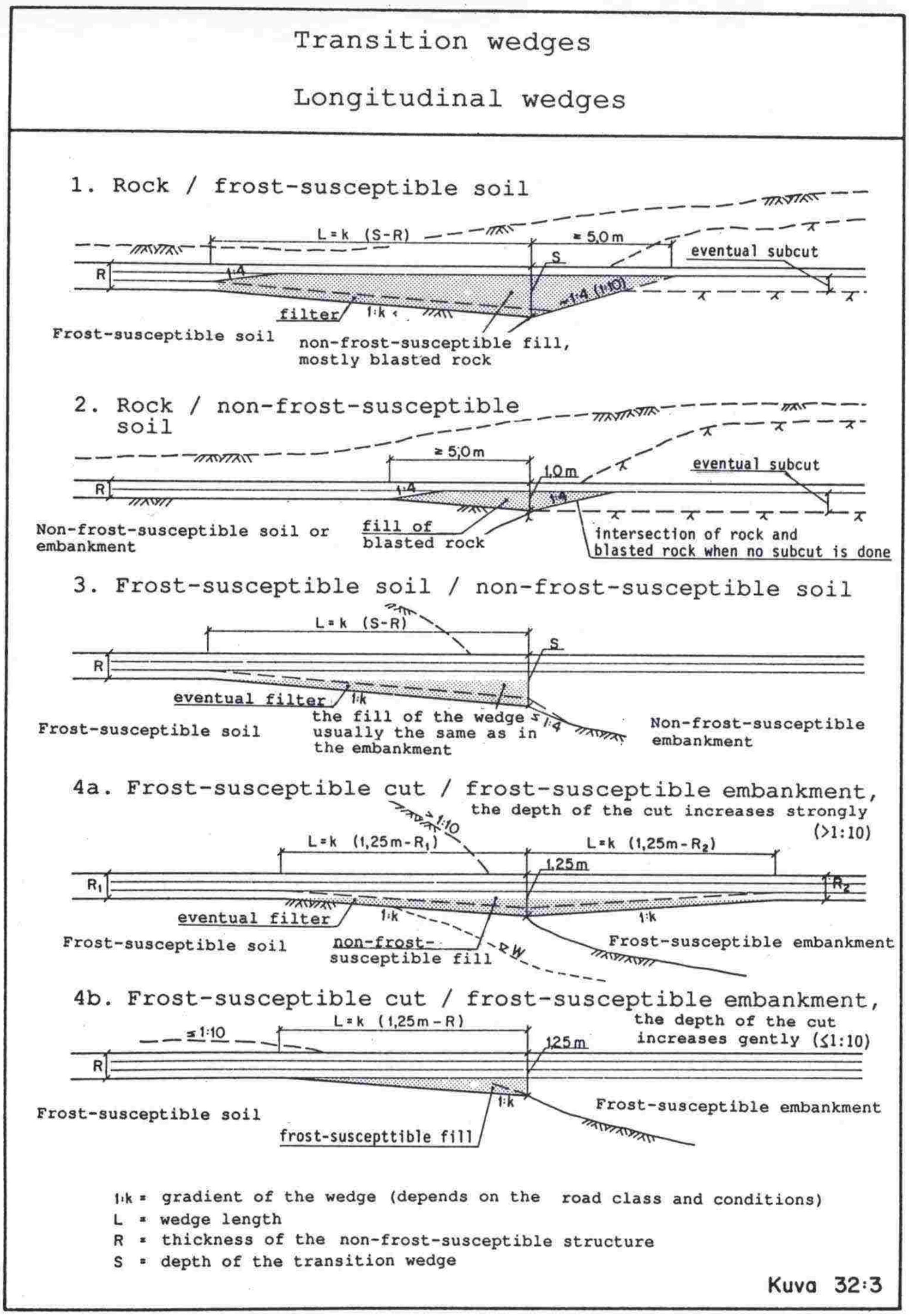
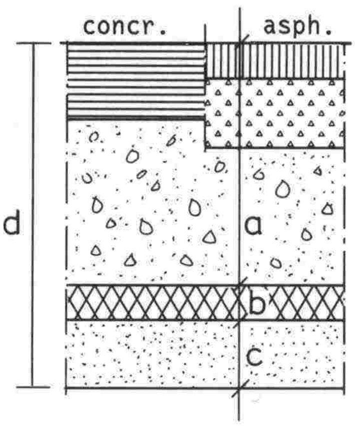


FIGURE B4-29. The structure of transition wedges according to the instructions of the Road and Waterways Administration /16/

b) The transition wedges can be compensated by thermal insulation or the frost penetration can be otherwise partly or totally prevented. The Finnish dimensioning principle for thermal insulation is discribed in Figure 30. In principle thermal insulations are suitable to be used in connection with concrete pavements. When the insulator takes care of the frost action problem, the structure can be designed on the basis of the load-bearing capacity and thus the rigid structure will be at its best /39/. When expanded clay is used, it evens possible differential settlements at the same time. Instructions have been drawn up to build base course of Styropor concrete or of other thermal insulating material among others in Germany and in the United States, /40, 41, 42/. In practice, the requirements of thick covering layers and the high prices of insulating materials restrict the use of insulators to transition wedges and other local objects, at least for the time being.

c) Also the design of drainage can considerably affect both frost action and frost damage. By locating the vertical alignment of the road as high as possible above the groundwater table the formation of ice lenses can be prevented. The drawdown of the groundwater table with open ditches or with subdrains may lead to the same result. A distance of 1,2 - 1,5 m between the vertical alignment and the groundwater table is demanded for example in the northern parts of the United States. The location of the vertical alignment or the drawdown of the groundwater table are not enough to diminish the risk of differential frost heave in the Finnish conditions of deep frost and variable groundwater table. Both measures are, however, important also here, because they reduce frost action damage and make the design of the pavement easier.

d) Besides the location of the goundwater table in relation to the vertical alignment also adequate drainage of pavement layers is important for the durability of the road in seasonal frost conditions. Inadequate drainage reduces the spring bearing capacity. Repeated freeze-thaw cycles - although they take place in non-frost-susceptible soils - cause pavement damage and irregularities during the course of time -especially if the pavement structure is water-saturated when freezing. As to the concrete pavement inadequate drainage of the surface strains joints and causes cracking. Intensification of the drainage of the pavement structure is an object of interest everywhere. Traditional open ditches are not considered sufficient any more, but the applicability of subdrains at pavement edges, Figure B4-31, is also contradictionary. Open pavement layers - also on slopes -can improve drainage whether or not subdrains are used. In any case efficient surface drainage can be considered essential for the long-term durability of the road no matter whether it is a question of asphalt or concrete pavements.



- the pavement structure on top of the insulation layer acc. to the load-bearing design so that the compressive stress of the insulation layer $\leq 1/3$ x the compressive strength of the insulation layer. $E = 30\ MN/m^2$ on top of the insulation layer assumed in Finland.

- to prevent slipperiness a sufficient amount of aggregate on top of the insulation layer, $a \geq 450$ mm in Norway, $a \geq 700$ mm in Finland

- an insulation layer of extruded polystyrene plastics or expanded clay (thickness ratio 1:6). When using extruded polystyrene the insulating thickness b is 30-100 mm in practice. If these thicknesses are not sufficient for complete frost protection, the thickness c of the filter course is increased under the insulation layer.

- the thermal insulated road structure aims mostly at complete thermal insulation. If frost penetration to the subgrade is allowed, the thickness of the filter course and that of the insulation layer can be diminished. In Finland c min = 150...200 mm.

- thermally 1 cm non-frost-susceptible material under the insulation layer is much more effective than on top of the insulation layer; hence the minimum thicknesses required by the load-bearing capacity and slipperiness are built on the insulation layer and possible additional thicknesses underneath.

THERMAL INSULATION THICKNESSES ACC. TO TVH

F 1000 x °C x h	d [mm]				
	850	950	1050	1250	1500
	b_o [mm]				
20-25	60	50	40	30	0
25-30		65	55	35	30
30-35		80	70	50	30
35-40		95	85	65	40
40-45			100	80	55
45-50			115	95	70
50-55			130	110	85
55-60			145	125	100

The table gives the necessary insulation thicknesses for thermal insulation in difficult frost conditions when the insulation layer is wet. In medium-difficult conditions $b = b_o - 30$ mm; when the insulation is dry $b = 0{,}85\ b_o$.

FIGURE B4-30. Design principles of the thermal insulated structure/3, 16/

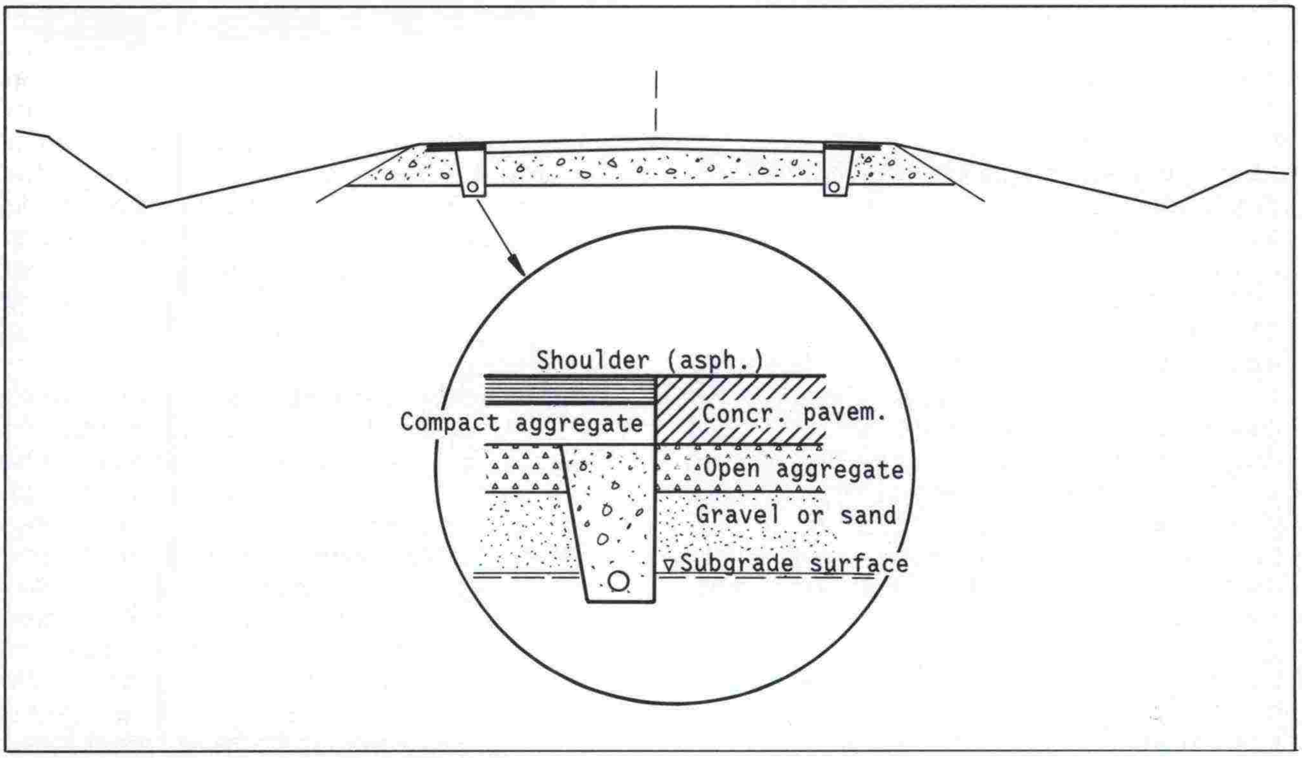


FIGURE B4-31. An example of the structure and location of subdrains

B 45 EXPERIENCES OF CONCRETE PAVEMENTS IN SEASONAL FROST COUNTRIES

B 451 Experiences from North America

Concrete pavements have been built also in frost-susceptible conditions in North America since the 1930s. The widest projects were executed in connection with the main road network in the 1950s and 1960s. All these concrete pavements are now worn out and most of them repaved. The durability and damage of these pavements have widely been analyzed and new design instructions have been made out on the basis of the experience. Frost heave is only one factor for the damage to concrete pavements and for the decrease of the service level - views of its magnitude vary in different states. Projects carried out on the basis of the new design instructions (shorter slabs, dowel bars) in the 1970s and 1980s are fewer and only short-term experiences of their durability is available. Main attention is paid to the rehabilitation of old slabs and to their repaving. Inspite of cutting down road investments the knowledge of concrete pavements is still kept up in most northern states.

Positive experiences of the durability against frost are reported from Minnesota, from the mid-part of the United States (freezing index 20000 x 30000 °C x h). Concrete pavements have been built on the Interstate highway network and on main roads in the surroundings of Minneapolis. New concrete pavements are under construction all the time, /32, 34/.

There are about 550 km of concrete pavements in corresponding conditions to those of Northern Finland (freezing index about 45000 °C x h) in Manitoba in Canada - in the adjoining state of Minnesota. Old pavements are mostly 15 -20 cm thick, unreinforced slabs of 6 m on an unbound base. The new standard pavement is 25 cm thick, the slab is unreinforced with skew and random spaced joints (3.6 m, 3.9 m, 5.1 m, 5.4 m) and without dowel bars. Approximately 13 km of new pavements are built annually. Separate frost bumps may appear at culverts or where the homogenizing has been badly carried out. Concrete pavements become irregular in the course of time and drivers are not always satisfied. Because concrete pavements are reliable and durable structures as to their load-bearing capacity, the present concrete roads will be preserved as concrete roads by developing rehabilitation and repaving methods, /31/.

Frost and rainfall conditions corresponding to those in Finland are prevailing also in Wisconsin - in the eastern neighbour of Minnesota. There are many concrete pavements of different ages in a different condition (about 10000 km). Half of the concrete pavements are unreinforced, and the slabs are not equipped with dowel bars. The latest pavements have been built with skew joints and with random slab lengths. In connection with the research journey we saw pavements in a good condition but also pavements that had failed already in the construction phase. New pavements are being built and old pavements are being repaved all the time. North of the capital Madison a modern two-lane pavement was photographed, Figures B4-17, B4-18, in the joints of which there were regular bumps of 2 - 3 cm apparently caused by frost heave, /32/.

There is a long construction tradition of concrete pavements (about 15000 km) in the state of Michigan. The applicability of the concrete pavement in severe conditions is, however, continuously discussed due to reduced evenness in the course of years./33/

Not very many concrete pavements have been built in the cold regions (>10000 °C x h) of the northeastern states, New York, Vermont, New Hampshire and Maine. According to the authorities the reason for this reluctance is the risk for differential frost heave. Another reason is the lack of high traffic density in these regions.

Concrete pavements have been built only in southern parts of Ontario in Canada, where the conditions are equal to those in Southern Finland or even milder. No frost action problems are reported from Ontario. New concrete pavements have not been built since 1974 due to lack of suitable projects. No concrete pavements have been built in the northern parts of the state due to the cold climate, /32/.

There are about 600 km of concrete roads in Quebec, mainly in the surroundings of Montreal where the freezing index is equal to that in Southern Finland. 500 km of these roads have been built before 1970 and they are now mainly paved with asphalt or otherwise reconstructed. 35 km of new concrete pavements have been built in the 1980s. Also the new pavements have caused problems as to the service level and damage, the public opinion has turned against concrete pavements. The difficulties are related to the stepping at joints of old pavements but also to the bad construction quality due to diminishing craftsmanship. A public committee has studied the condition of concrete pavements and the future pavement policy in 1987 and recommended short slabs with right-angled joints and equipped with dowel bars as a type of new pavements. The committee also recommends a 5-year-programme to preserve the craftsmanship. It also seems possible that Quebec won't build any concrete pavements for the time being, /30/.

The researchers of the US Army Cold Regions Research and Engineering Laboratory, C.R.R.E.L, have a clearly reluctant attitude towards concrete pavements in cold regions. In their opinion homogenizing - widely used in North America - is not sufficient to prevent differential frost heaving and detrimental unevenness. If, on the other hand, a thick non-frost-susceptible pavement is built, the concrete pavement loses its competitiveness in traditional road projects. The alternative structures described in Chapter B 4422 can be used as a technical alternative according to the researchers, but in practice they recommend unbound layers and asphalt surfaces when traffic flows are not very great. The researchers of CRREL emphasize, however, that their point of view is primarily geotechnical /32/.

According to the American Conrete Pavement Accociation, ACPA, concrete pavements are suitable also in cold climates. ACPA makes out semi-official design instructions and they are engaged in wide training activities all over the United States. Unreinforced short slabs equipped with right-angled joints and dowel bars are recommended as a pavement type in cold regions, /32/.

B 452 Experiences from Europe

The frost penetration depth is generally not more than 80-100 cm in Central Europe and at least the main roads are built down to the non-frost-susceptible depth; thus actual frost damage is not reported. There are long-term concrete road traditions also in Central Europe; old, worn-out pavements can be found. The damage of these pavements is partly due to frost action but frost action is not the main pavement-straining factor. Instead, repeated freeze-thaw cycles may cause cracking and scaling on the pavement.

In the Austrian and Swiss Alps concrete pavements have been built on motorways and their high-class pavement structure can sufficiently prevent also frost damage. Thus, as to frost action, concrete pavements have very well been established in these countries. Dowel bars are necessary to ensure a durable evenness. As to the serviceability the best concrete pavements can be found in Switzerland and Austria, /35/.

There are not many concrete pavements in Scandinavia and thus experiences of the sufficiency of the frost design are inadequate. Oldest pavements of the 1930s have been repaired partly because of frost damage. Thus attention has been paid to the frost protection design and the latest concrete pavements have quite well been preserved and frost action has not reduced the service level. Single frost damage can be found on present concrete pavement sections: for example in Norway a section of about 40 m on the road section Klinestad - Langåker on the Main Road E18 built in 1979 has apparently been destroyed due to frost action. Both frost heave and a decrease in the spring bearing capacity at the intersection of a sloping terrain and a cut have been exceptional and cracking has quickly advanced. Other parts of the road are in a perfect condition, /36/. A corresponding but a milder damage has taken place at the intersection of an embankment and a low rock cut on the Kalkkitie in Parainen, Finland (built in 1982, Figures B4-32, B4-33).

Frost action is regarded as a partial reason for the damage on concrete pavements built in the 1960s in Malmöhuslän in Sweden. This is one of the reasons which have resulted in the opinion in later cost comparisons that a concrete pavement causes additional costs in frost protection, /37/.

FIGURE B4-32. Frost damage on Kalkkitie in Parainen (concrete pavement from 1982)

FIGURE B4-33. Frost damage on Kalkkitie in Parainen, a slab damaged by frost

B 46 SUMMARY OF CONCRETE ROADS IN SEVERE CLIMATES

Concrete and asphalt pavements are equally included in the official design instructions all over Europe and North America. Both ways are suitable for structural design also in severe circumstances. As a rule, the requisite pavement thicknesses are of the same size range for both pavement types; according to some instructions thinner pavement layers may be built when concrete is used. Today the American design practice is cautious with the use of concrete in cold seasonal frost conditions. This results from the irregularity of old pavements due, at least partly, to repeated differential frost heaving. However, in evaluating the American situation the following points should be considered: lack of actual frost protection (the homogenization of the subgrade soil is counted upon), old-fashioned design of the slab from the European point of view, and a low strength concrete. Unevenness caused by frost action is not known in Central Europe or it has been overcome by modern design. Cold climatic conditions, however, reduce the competitiveness of the concrete alternative to some extent; the good load-bearing capacity of concrete pavements will be better utilized in non-frost-susceptible circumstances.

From a general point of view the frost conditions are more difficult in Scandinavia than in the rest of Europe and the Finnish conditions are the worst in Scandinavia. When calculated per project, however, only 10 - 20 % of the road length must be dimensioned to the extreme frost conditions, for the main part of the network length the location of the vertical alignment, subgrade soil type etc. keep frost damage moderate. In any case the frost protection practice applied elsewhere should not directly be followed in the severe Scandinavian conditions. Generally, concrete pavements are not included as an equal alternative to asphalt pavements in the official Scandinavian design instructions. There are unofficial instructions available for the design of concrete-paved road structures in Sweden and Finland; the official design instructions are not sufficient even in Norway and Denmark. It seems that the frost protection has been solved temporarily, on different grounds in different Scandinavian countries. Finland has adopted the most cautious line of action; Norway and Sweden design all their road structures with a greater risk for frost action.

The following answers to the questions asked in Chapter B 40 can be given as a summary:

1) The rigidness of the concrete pavement cannot be utilized as a reduced pavement structure thickness in the Finnish conditions. Instead, in thermally insulated structures the concrete pavement opens new possibilities also in the thickness design. On the other hand, there are no grounds for the belief that the concrete pavement would demand a thicker frost protection than the asphalt pavement. Hence, as to the frost protection both pavement types are equal. Whether the current Finnish frost protection design procedure should be revised is another question.

2) The concrete pavement has every qualification to preserve its evenness also in frost-susceptible conditions in the long run if the structure is properly designed. The joints are always the weak point of the concrete pavement. Slab movements, water and ice strain the joints greatly in the seasonal frost conditions. Thus a good service level requires special attention to the joint design, work performance and maintenance.

3) Concrete pavements should be designed as short slabs (about 4 - 5 m) and equipped with right-angled transverse joints and dowel bars in frost-susceptible circumstances. This design principle has already been adopted in Finland; foreign experiences confirm that it is a right solution in the Finnish conditions.

4) According to foreign experiences concrete pavements have preserved as well in frost-susceptible circumstances as in warmer conditions. Repeated freezing and thawing is a stress factor which together with other factors will fatigue the pavement and the entire road structure. As material, concrete can endure the most severe circumstances. As a pavement concrete is the most reliable structural alternative as to its load-bearing capacity and service level also in seasonal frost conditions.

CHAPTER B4 CONCRETE ROADS IN SEVERE CLIMATES

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CHAPTER B5
CONCRETE PAVEMENTS ON WEAK AND COMPRESSIBLE SOILS

CHAPTER B 5

CONCRETE PAVEMENTS ON WEAK AND COMPRESSIBLE SOILS

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B 5
CONCRETE PAVEMENTS ON WEAK AND COMPRESSIBLE SOILS

B 50 INTRODUCTION

The level of the road surface is not permanent. In addition to frost heave also settlement and deformation caused by different reasons reduce the evenness of the road surface and strain the durability of the pavement in the course of time. Embankments often become even more compact than after the paving, permanent settlement may take place during thawing and the fatigue of the pavement or the pavement structure under traffic loads may cause rutting or settlement on the road surface. In Finland the most significant settlement risk, however, is caused by the consolidation of the road bed soil.

Settlement can be prevented or at least levelled by good design and high-quality construction work. However, it is often inappropriate to build roads insensitive to differential settlements when long road sections lie on deep compressible soils. The embankment will then be built just on compressible soil and differential settlement is to be expected. In Finland this expectation means that cosmetic and even traffic faults are endured to a relatively great extent, only clearly detrimental differential settlement and pavement damage are repaired. Thus a functional vertical alignment, deviating from the original one will be formed to the road syrface in the course of years.

As to asphalt pavements settlement can be levelled when the pavement is reconstructed due to wear or other reasons. Regarding concrete pavements a repeated levelling is more difficult, but, on the other hand, the concrete pavement as such prevents and levels deformation of the road structure.

The behaviour of concrete pavements on weak and compressible soils arouses many interesting questions:

- What kind of differential settlement can be allowed without breaking the concrete pavement itself ?

- Can the service level of the concrete pavement be preserved good also on compressible soils during the entire service life ?

- Do concrete pavements require more expensive subgrade strengthening measures than asphalt pavements to restrict settlements ?

- How settlements of concrete pavements can be repaired ?

- Can the rigidity of the concrete pavement be of benefit to levelling differential settlements ?

These questions will be dealt with on the basis of Finnish and Swiss experiences in the following. Internationally, concrete pavements are regarded as pavements suitable only for subgrade soils insensitive to differential settlements. Thus active interest in the study of the settlement of concrete pavements has been shown only in countries where they are generally constructed and where much compressible subgrade soil is to be found. In addition to Switzerland and Scandinavia such countries are at least Holland, certain states in the Soviet Union, Canada and in the United States.

In order to get a general view of the significance of the settlement issues in Finland also causes for settlement and the Finnish design practice are dealt with in the following.

B 51 BASIC FACTS ABOUT SETTLEMENT AND SETTLEMENT DAMAGE ON ROADS

B 511 Causes for and consequences of road settlement

A load which strains the subgrade or the road structure is the cause for consolidation or differential settlements. The load can be:

- the weight of the embankment on a compressible soil or on a soil liable to slides
- the drawdown of the groundwater table in connection with drainage of the road
- the weight of the embankment itself on a badly compacted embankment or on an embankment loosened by frost
- a traffic load as a compacting or fatiguing factor of the structure
- dynamic impacts of traffic flows

This results in differential settlement of the road surface and in a decrease of the service level of the road in the course of time. Settlement due to the compaction of the embankment or traffic loads can be controlled by means of construction design. Instead, settlement caused by the consolidation of the subgrade soil must be studied by soil mechanics.

B 512 Consolidation of the subgrade soil

Total settlement of the subgrade soil due to the road embankment load can be described with the following formula /12/:

$S = S_a+S_k+S_d+S_j$
S = total settlement
S_a = initial consolidaton
S_k = primary consolidation
S_d = settlement due to deformation
S_j = secondary consolidation

Initial consolidation S_a (= elastic consolidation) takes place in the subgrade soil immediately in connection with the embankment construction and, in any case, during the road construction; thus, as a rule, it has no importance in the settlement study later on. Settlement due to deformation S_d can be significant, if the shearing resistance of the soil is low or if the strength of the embankment against slide is low, but, however, S_d is ignored in normal safety factors of 1,5 - 1,7 in settlement calculations - also because there is no reliable method for its mathematical definition. Thus, settlement of the embankment as a time function is considered to result from actual consolidation S_k and secondary consolidation S_j (Figure B5-1).

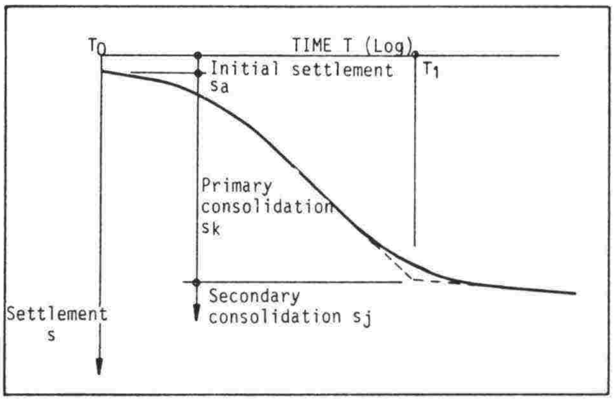
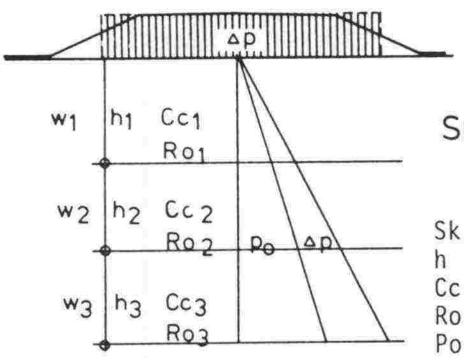


FIGURE B5-1. Time consolidation curve of compressible soils (additional load p, at the moment To)

Consolidation of a fine-grained soil layer is usually the greatest part of the total settlement in magnitude. The magnitude depends on the size of the additional load, depth of the weak soil and on the compressibility properties of the soil deposits (Figure B5-2). The compressibility is defined from undisturbed soil samples with an oedometer. Because it is known that compressibility is directly proportional to the moisture content of the soil layer, the magnitude of the consolidation can roughly be determined also by the moisture content (Figure B5-3). /11, 13/



$$Sk = \sum_{1}^{h} h \times \frac{Cc}{1+Ro} \times \log \frac{po + Ap}{po}$$

Sk = consolidation settlement
h = layer thickness
Cc = compressibility coefficient } defined in a
Ro = original void ratio } oedometer test
Po = effective stress
Δp = additional load

FIGURE B5-2. Calculation of consolidation settlement SK /11/

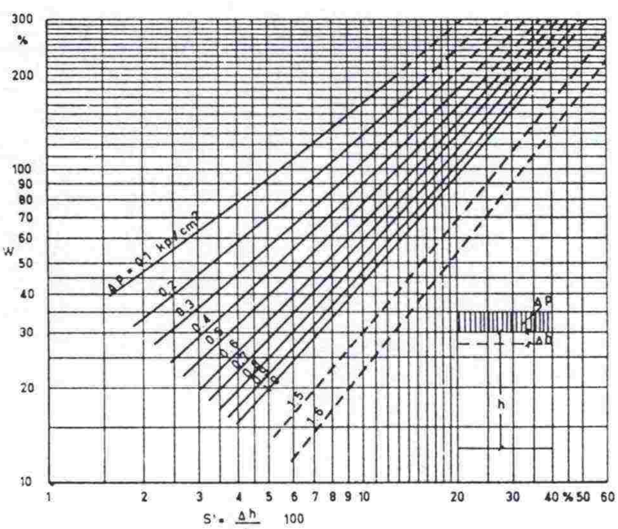


FIGURE B5-3. Relative settlement as a function of water content and load /11/

As qualifications of soil layers and the effective additional load vary in the upright direction, total settlement is defined as a sum of the differential settlements calculated layer by layer. As a rule, settlement calculations at individual spots are nowadays made by means of computer programs and they give relatively reliable forecasts of the magnitude of settlements. The time dependence of consolidation is defined by means of an oedometer test (Figure B5-1). Consolidation of coarse-grained soils takes place quickly due to good water permeability, but consolidation of fine-grained soils (clay, gyttja, peat) occurs in the course of years to the degree that water is running away from pores and the pore-water pressure is decreasing. When taking into account also the secondary consolidation S_j, the settlement of the embankment with ground clearance can be considered to continue on weak soils of clay, gyttja and peat during the entire service life of the road. There are practical examples that total settlements can be as great as 1 - 2 m.

B 513 Differential settlements of the subgrade soil

For the definition of differential settlements along the road alignment the above investigations and calculations at various spots must be carried out and the settlement profile of the road investigated. An exact definition of differential settlements is a troublesome and difficult task. Although the depth of the weak soil and the height of the embankment are constant, differential settlements will vary on different road sections due to variations in the moisture content. When taking into account embankment heights and alterations in the depth of the weak soil layer the evaluation of the settlement profile is an even more difficult task. As a rule, a design based on a few calculations at various spots is regarded as sufficient.

B 514 Inconveniences due to settlements and repair measurements

The damage caused by settlement directs at:
- the outlooks of the road
- driving comfort
- driving safety
- the durability of the structure

Even small repeated settlements, especially on high-quality roads, can be considered at least cosmetic faults. The driving comfort will reduce if the driving speed has to be lowered due to impacts at vehicles. As to driving safety for example, transverse rutting of the road surface, gathering of water on the road surface or restricted sight distance due to settlement may be fatal. As to the pavement structure the abrupt settlements for example at culverts will be most straining. Cracks, pop-outs and faulting can be produced on the pavement. Intersections of a settled embankment and an undepressed structure like those of a bridge and a rock cut are critical.

Settlement does not necessarily cause any damage if it takes place evenly and for a sufficiently long distance. The magnitude of settlement is not essential as such, instead, differential settlements and their slope in the longitudinal and transverse direction are important. Differential settlements can be measured as a tilting speed W (o/oo), Figure B5-4.

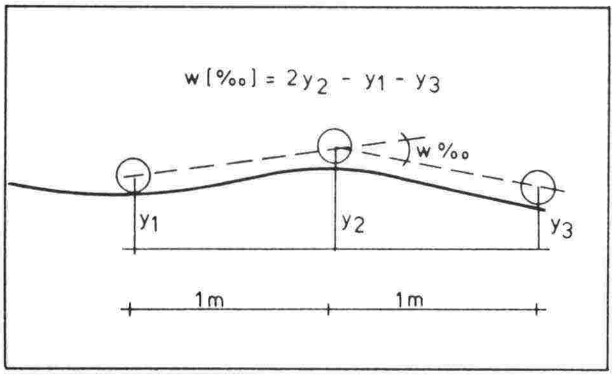


FIGURE B5-4. Design principle of the tilting speed w (o/oo) /15/

Slopes and slope changes are determined by levelling at intervals of one or two metres and they are compared with recommended values defined by tests, Figure B5-5. /11, 15/ The relation of the settlement depth and length can be investigated correspondingly, Figure B5-6. The damage can be estimated on the basis of the recommended values or on the basis of the change in the driving dynamics of the design vehicle (quantity k in Figure B5-6).

Functional class	Allowed slope change (o/oo)	
	Minimun quality level	Maximun quality level
Motorways	6	4
Main roads I and II	8	5
Regional roads	11	7
Collector roads	16	9
Connecting roads	-	15

FIGURE B5-5. Allowed longitudinal slope changes /11/

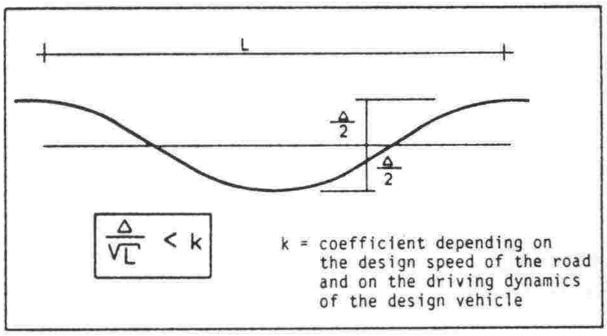


FIGURE B5-6. The depth/length ratio of the settlement as a detriment rate /1/

B 515 Allowed settlement and the need for settlement repairs

Roads under traffic loads are not completely even, but there are settlements on them - at least to some extent - caused by different reasons. The extent of the damage degree endured influences not only the construction costs of the pavement structure and subgrade strengthening but also maintenance costs. The road category and design speed are the most important variables in defining allowed settlements. The numerical values in Figure B5-5 are adopted design values in subgrade strengthening in Finland. They are suitable for a general project general study of settlement situations both for asphalt and concrete-paved roads. Consolidation-induced damage in details depends on so many factors, as described above, that the allowed consolidation and differential settlements of a single road section are determined in the course of design.

In general, in Finland no attention to the consolidation of a road under traffic loads is paid until the driving comfort is affected. When repair actions are taken, consolidation is generally greater than that in Figure B5-5. The repair action may include levelling the pavement with asphalt or wider repairs of the structure. A new vertical alignment may be designed for the repair; settlement repairs do not generally aim at the original vertical alignment of the road.

B 516 Prevention of consolidation damage

The object of the road design is to produce as uniform and durable a road structure as possible conformed to varying conditions of terrain, soil and traffic. Settlement of the subgrade soil is one threat for the long-term service level of the road. Although it is not always possible to produce a road which is not liable to differential settlements, risks for detrimental consolidation can be minimized by careful design and construction, often without special additional costs. Preventive measures of this kind are at least:

1) Means of geometrical design

The location of the horizontal and vertical alignment can decisively effect on the formation of consolidation settlements. The horizontal alignment must avoid abrupt transitions from firm soil to weak soil. The aim of the vertical alignment is a low and even embankment height and a gentle transition between weak and firm soil. The embankment height must be so adjusted that a good security against slide can be achieved (coefficient at least 1,5 - 1,7). Thus deformation, which is not easily controlled, can be avoided.

2) Means of structional design

Addition of the rigidity of the pavement structure decreases local differential settlements and reduces the sharpness of the grade line alterations no matter whether they are due to deformation of the subgrade soil or embankment. A more rigid structure also retards changes in the line of maximum slope of the road surface. Coarse pavement materials, thicker bound layers and concrete pavements promote this object. Not only the rigidity of the pavement slab but also material choices of the entire pavement structure and first-rate performance of the construction work are important in minimizing differential settlements. Pavement structures on compressible soils are built as sandwich structures in Switzerland and Holland, Figure B5-7. A cement-treated layer as a lowest layer ensures the rigidity of the entire pavement structure as the unbound subbase can more effectively be compacted. A cement-treated layer built on the lower part of the subbase could correspond to the sandwich structure in Finland (Figure B5-8).

3) Subgrade strengthening measures

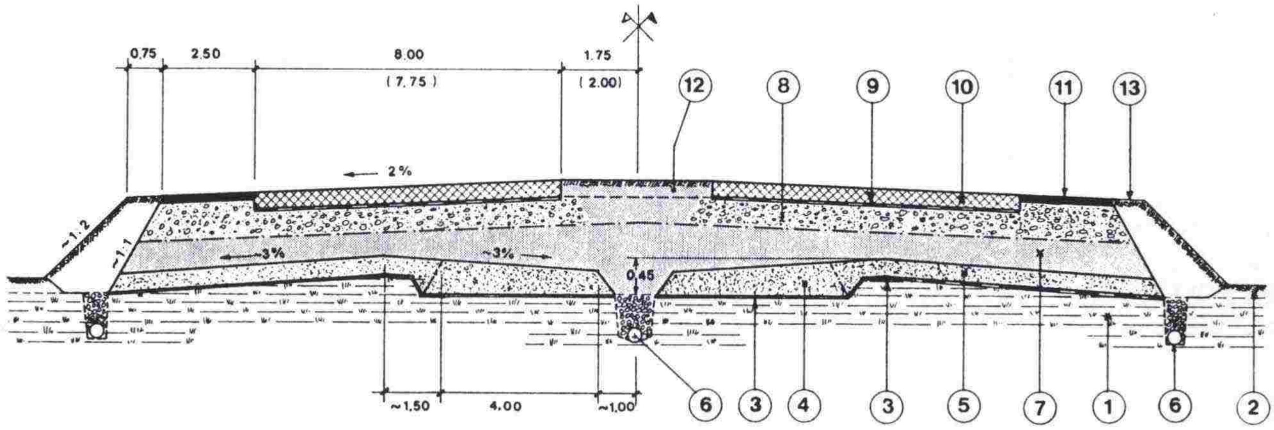
If geometrical or structural measures do not result in a sufficiently good outcome in minimizing differential settlements the consolidation of the subgrade soil can be affected by special subgrade strengthening actions. Most typical measures are shown in Figure B5-9:

- replacement of soil
- light-weight embank ment material
- deep vertical drainage with strip
- deep lime stabilization
- embankment piling
  and combinations of them especially in transition structures

Of the above methods the replacement of soil is a natural and profitable strengthening method when the strength of the subgrade soil is low, the replaced amount is reasonable and there are substituting materials available.

It is possible to build floating embankments on compressible subgrade soils using expanded clay and other light-weigth fills, even so that the subgrade is not exposed to any extra load. Light-weight fills are effective to restrict consolidation settlements; their use has increased year by year, especially in transition wedges.

Normalprofil für N3 SG Flums—Mels



(1) compressible subgrade
(2) organic topsoil
(3) polyethene sheet
(4)(5) cement-treated layer
(6) subsurface drain
(7) fill (0 - 60 cm)
(8) subbase (40 cm)
(9) polyethene sheet
(10) concrete pavement (22 cm lightly reinforced)
(11) shoulder pavement (asph.)
(12) median
(13) shoulder and slopefill

FIGURE B5-7. Sandwich structure on a compressible embankment, a Swiss example /9/

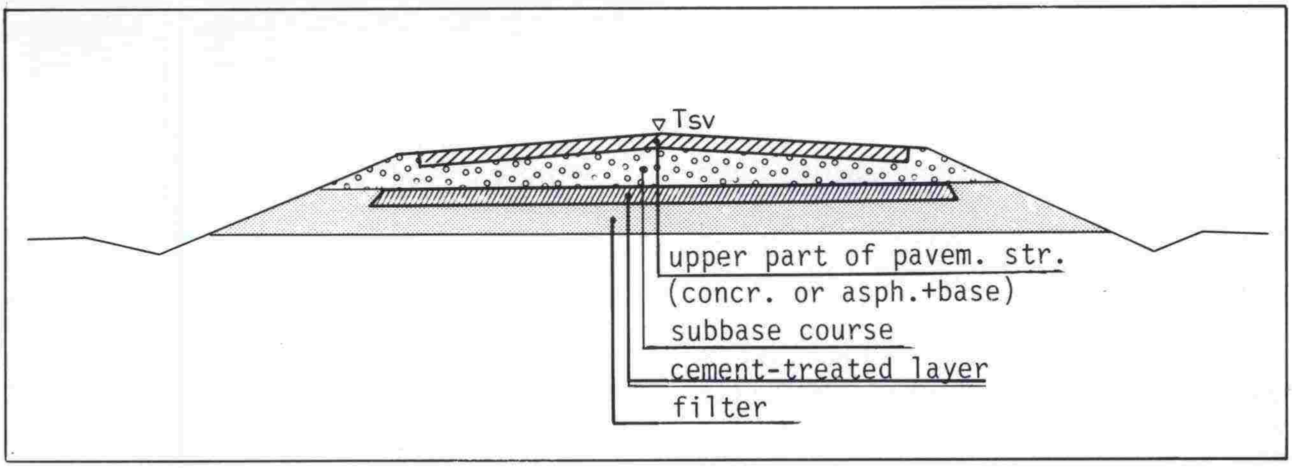


FIGURE B5-8. Suggestion for the Finnish sandwich structure on a compressible soil

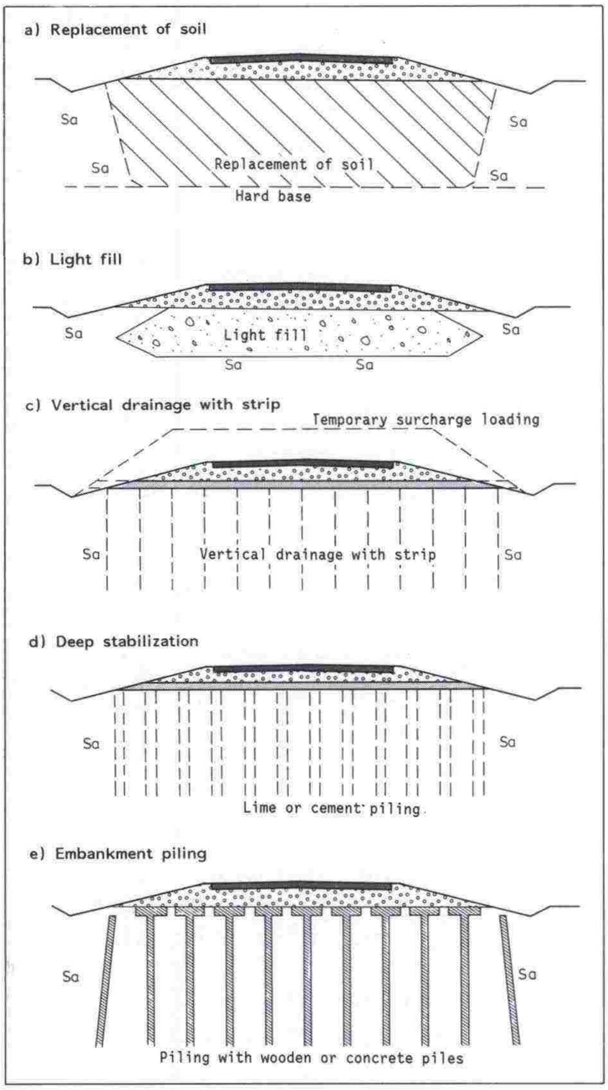


FIGURE B5-9. The most common subgrade strengthening measures

Permanent deep vertical drainage is a reliable, effective and profitable method to accelerate consolidation on deep compressible soils. It is possible to produce almost all consolidation before paving the road and before opening it to traffic by resorting to permanent vertical drainage with strips and with temporary overloads. Thus the risk for differential settlements decreases decisively. After the diffusion of the method it has widely been adopted also in Finland in the past few years.

Deep stabilization has long been carried out as lime piling. The method improves the stability of the embankment and reduces total settlement. The lime piling is a widely-used method especially in the city street construction in Finland. Deep stabilization can be used also in soils where lime is not applicable by compensating lime partly or totally with cement.

Embankment piling is already a traditional method in the prevention of settlements; with piles the embankment load can be transferred through weak soil deposits onto the firm soil bed or rock. The most typical area of piling is a high embankment on deep compressible soil where the stability of the embankment does not allow building on the soil bed. Embankment piling is an expensive and inappropriate method in the prevention of consolidation on wide road sections crossing compressible soils.

Use of the subgrade strengthening methods is most demanding in their application to transition structures, Figure B5-10. It is possible to build the intersection of an undepressed road section - bridge, rock cut etc. - and weak soil using subgrade strengthening methods so that the good service level of the road can be preserved during the entire service life. This calls for thorough terrain investigations and careful geotechnical design. Transition structures are often considerably expensive and hence they are sacrificed quite often - at the expense of the decrease in the service level and the increase in settlement repairs.

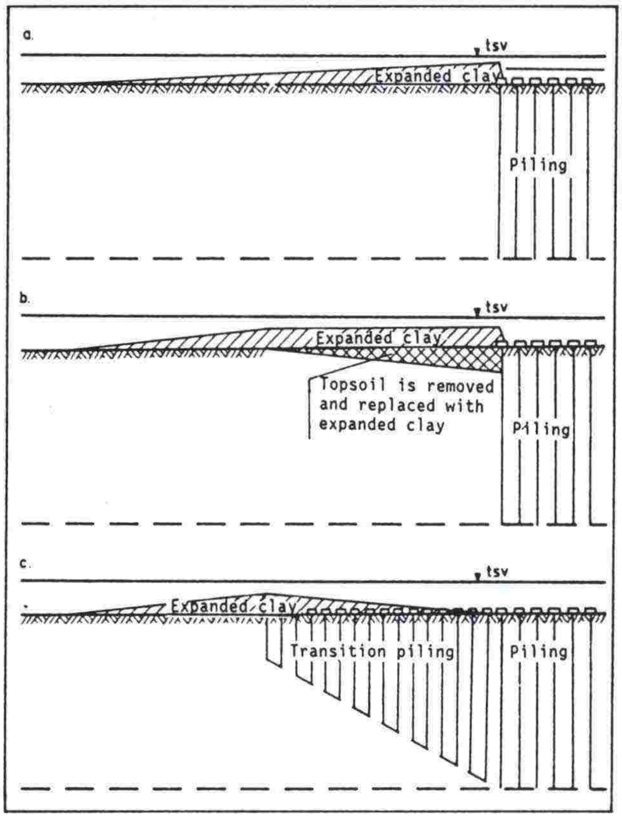


FIGURE B5-10. Different kinds of settlement wedges /11/

B 52 OCCURRENCE OF WEAK SOILS IN FINLAND

The post-glacial melting of the inland ice covered the main part of the present western and southern Finland under the sea. Fine-grained soils stratified (cedimented) into water during the sea and lake ages and now owing to the land rise they form the clay and gyttja layers on the coast of the Gulf of Bothnia and the Gulf of Finland, Figure B5-11. The average thickness of clay courses is 10 m, but especially in southern Finland clay areas can be wide plateaus in connection with rock exposures or moraine hills and their thickness can be tens of metres. The high moisture content and low strength are typical of Finnish clay courses due to their mode of origin and young age. Swamps are younger formations of soil which keep emerging mainly as a result of forests turning boggy. Water on water-soaked clay and moraine soils prevents the putrefaction of the vegetation which results in the formation of swamp peat. About 30 % of the area of Finland is covered by swamp. Peat courses are usually 2 - 3 m thick in northern Finland and 4 - 6 m thick in southern Finland.

As far as the consolidation risk is concerned the most difficult areas in Finland are seaside areas. Differential settlements become prominent because there are many rock exposures and because moraine courses are thin. In other parts of the country the consolidation risk is smaller and weak soil sections are shorter.

From an international point of view the Finnish mineral soils are geologically very young (less than 10000 years) and the moisture content of fine-grained courses very high and the strength poor. Thus it is prominently important to control stability and consolidation problems in the construction on weak soils in Finland. Clay courses corresponding to those in Finland can naturally be found on the coast of the Gulf of Bothnia in Sweden and on the coast of the Gulf of Finland and Baltic Sea in Baltia. Thick clay soils sensitive to disturbances are found on the Norwegian coast which geotechnically are very difficult. Weak soils are found also in other parts of the world; the design principles of their subgrade strengthening are similar to those described above, but the mode of origin and qualifications of the soil deposits are different to those in Finland. All over the world roads are more and more often built on mariginal subgrade soils unsuitable for any other purposes. That is why consolidation problems are to be studied more and more thoroughly in most countries.

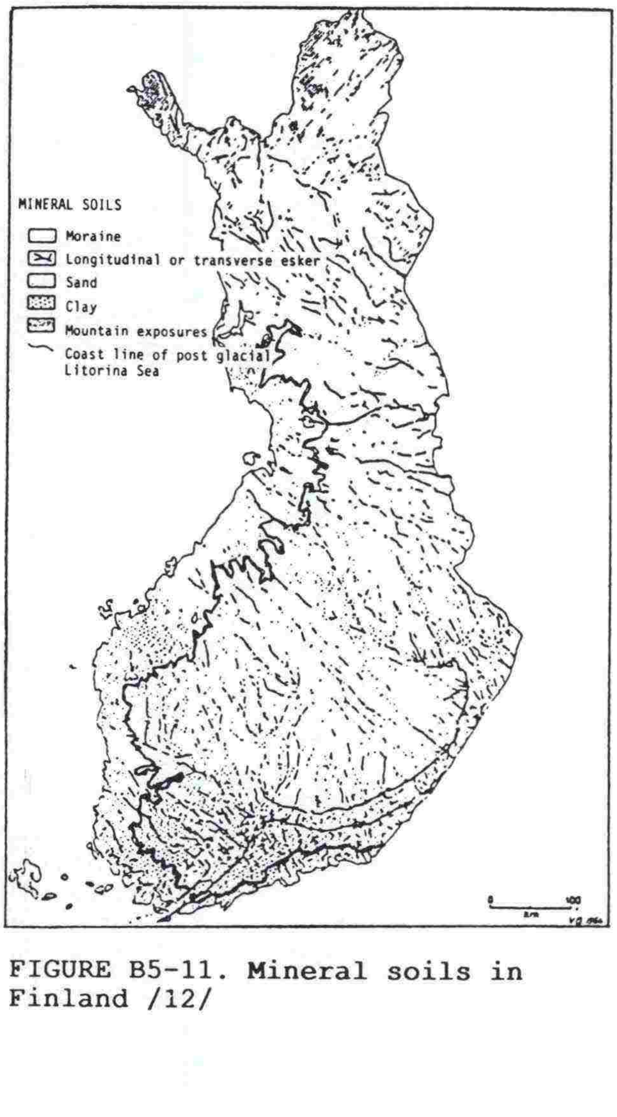


FIGURE B5-11. Mineral soils in Finland /12/

B 53 SETTLEMENTS AND CONCRETE PAVEMENT

The above survey has studied the environment in which the pavements are designed in Finland. The survey has been made neutral on purpose; the basic data of consolidation are not related to pavement types. The main part of the Finnish roads are built on soils insensitive to differential settlements, but roads on swamps and on clay on the coast are to be sensitive to differential settlements. These roads will settle in the course of time and this will result in a reduction of the driving comfort and in certain maintenance costs. Finland has good experiences of the usage, durability and behaviour of asphalt pavements on compressible soils. The experiences of concrete pavements are, however, restricted to few results from test roads. Hence, also the available foreign investigations and experiences are resorted to in the following study of the advantages and disadvantages of concrete pavements on weak and compressible soils.

B 531 Advantages of conrete pavement

It is a well-known fact that the great rigidity of the pavement structure will even out differential settlements. However, a mathematical demonstration of this fact has been proved difficult. Methods for the consideration of the rigidity based on the investigations at Newark Airport have been developed by Mr Nai C Young from the United States /1/. His general conclusions are:

- The rigidity of the pavement contributes to a reduction of the reflection of differential settlements as irregularities on the road surface, although they cannot entirely be prevented.

- The rigidity of the pavement increases the wavelength of settlements and thus minimizes the damage of differential settlements.

- A correctly designed rigid pavement restricts also total settlement /4/, since the stresses of the subgrade due to traffic loads remain small.

- Contrary to a flexible structure the rigid pavement endures stresses at the pavement caused by consolidation and forms "bridges" over local differential settlements. Not until the relation between the depth and length of the settlement is too great will the slab break down and the road surface follow the consolidation line of the soil.

- Also the concrete pavement "bends"; the creep rate E^1 is about 1/3 of the E-modulus of concrete.

On the basis of mr Yang's study it can be stated that if total settlement is reasonable (with or without subgrade strengthening) a rigid pavement can even out differential settlements so that no measures are needed to improve the regularity of the pavement during the service life of the road.

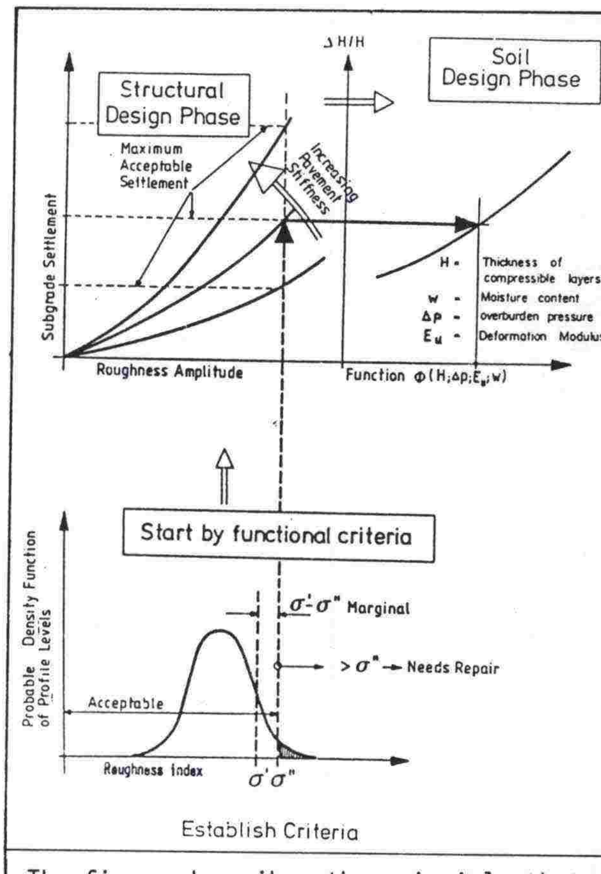
A well-known Swiss concrete pavement expert Mr Willy Wilk agrees with the opinions of Mr Yang and he has continued investigations on the advantages of the rigidity in the pavement design /2,3,4/. In Mr Wilk's opinion not only the pavement thickness but also the rigidity of the entire pavement structure is to be determined on the basis of soil properties and evenness requirements when a road structure sensitive to differential settlements is designed, Figure B5-12. Mr Wilk points out that a cement-treated layer on the lower part of the pavement structure is an important advantage on weak and compressible soil (compare with Figure B5-7):

- The cement-treated layer improves rigidity, but it also

- improves compaction qualities of unbound layers on it and thus

- it ensures the highest possible E-modulus for all layers

Mr Wilk states further that on weak and compressible soils a better result and a longer durability are obtained with concrete pavements and with pavement structures which are as rigid as possible than with asphalt pavements. Thus Wilk recommends concrete pavements as pavements on weak soils in the Swiss conditions. It is provided, however, that primary consolidation is mainly produced by surcharge loading or deep vertical drainage before paving.



The figure describes the principle that the same evenness of the road surface allowes the greater total settlement the rigider the pavement structure is.

FIGURE B5-12. Stiffness design principle of the pavement structure on compressible soils (W. Wilk, Switzerland) /12/

B 532 Risk factors related to concrete pavements

Subgrade soil consolidation is a risk for the service level and durability of the road irrespectively of the pavement type. When designing pavements for conditions sensitive to differential settlements these risks have to be recognized and, if possible, affected by means of design. Concrete pavements are prone at least to the following risks:

1) Danger for cracking

When too great a stress is induced by differential settlements as to the tensile strength of the slab, the slab breaks. This kind of a danger is evident in the vicinity of culverts and bridges. A longitudinal crack may develop when road edges consolidate more than the mid-area of the road due for example to poor compaction. It is usual that stresses at slabs discharge at joints and possible cracks remain as hair cracks which do not hazard the load-transfer nor regularity. Wild cracks will develop on concrete pavements for many reasons, often the consolidation is not the most important factor. When necessary, cracking can be prevented with reinforcement of the slab or by making transition structures, Figure B5-13.

2) Danger for joint damage

Angle deformation in joints and simultaneously tension or compression at their upper and lower edges will occur on weak and compressible soils, Figure B5-14. Compressive stresses can break the slab at the joint, tensile stresses tend to open the joint. Joint damage can be prevented by building a wider saw-cut or using joint sealant and a joint former strip at the bottom of the slab.

3) Faulting or stepping at joints

It is thinkable that the rigid pavement "collects" deformation due to consolidation at joints where they occur as detrimentally great angle differences or faulting. According to the Swiss and Finnish experience (Figures B5-19, B5-22) the concrete pavement has time to creep during consolidation and no detrimental angle differences will occur, although total settlement may be great. Requirements provided by the road category (compare Figure B5-5) have to be set for angle differences between slabs. Dowel bars seem to be necessary in pavements sensitive to differential settlements.

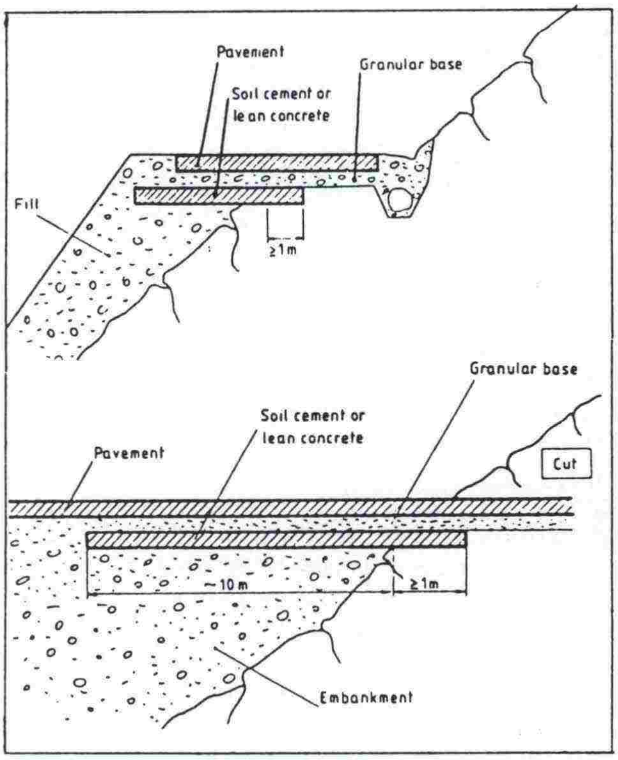


FIGURE B5-13. Evening out differential settlements with soil-cement wedges (W. Wilk) /12/

They prevent faulting from occurring between slabs, although deformation of the adjoining slabs would be considerable. It has been proved in France that a pavement built without dowel bars will loose its service level quickly (Figure B5-21).

4) Danger for ponding on an old pavement

The road consolidates more at the centre than at the edges. Crossfall will decrease in the course of years and finally there will be ponds on the road. The change of the crossfall can be retarded by reducing the total settlement in advance, stiffening the road structure in the transverse direction or - when it is a question of concrete pavements - by taking measures for the drainage of ponds. Instructions on drainage by means of transverse grooves are found in the Swiss standards. The grooving principle has been presented in Figure B5-15. The transverse grooves in the figure are always needed already on a new pavement when the gradient of the surface is becoming too small regarding the flow of water. The ponded area can, of course, be repaired by lifting slabs or by additional overlay. Local concentrations of water can be removed also by milling.

5) The long service life of concrete pavements and difficulties in repairing consolidation damage

The long service life of the concrete pavement - perhaps 3 times as long as with asphalt pavements - means that there is time for occurrence of more consolidation or differential settlements and they are not meant to be repaired during the service life. Even for this reason the magnitude and speed of the consolidation phenomenon must be restricted by subgrade strengthening methods. Also the threshold of concrete pavement repairs have been kept higher and the repair measures considered more expensive than those of asphalt pavements. The need for repairs can be minimized by subgrade strengthening measures and by the good quality of the work at all construction stages. However, if damage, which must be repaired, occurs on the conrete pavement, it is taken care of as follows:

- Faulting in joints or on the slab are locally milled.
- Bad damage where slabs have been broken (for example slope failure or a steep break at a piled culvert) is repaired by reinforcing the base and making new slabs.
- Joint or crack damage is taken care of by traditional repair methods of concrete pavements.
- Detrimental settlement where consolidation is still going on are temporarily repaired with asphalt.

When the age and need for repairs are studied from the above points of view of Mr W Wilk, a principal picture of the reduction in the service level of roads in the course of years can be presented, Figure B5-16. Because rigid pavement structures even out differential settlements better than asphalt pavements - although they do not reduce total settlement - a lower decrease in the service level is achieved. Thus the concrete pavement, inspite of its long service life, offers a better service level than the asphalt pavement also on weak and compressible soils when both pavement types have the same subgrade strengthening measures.

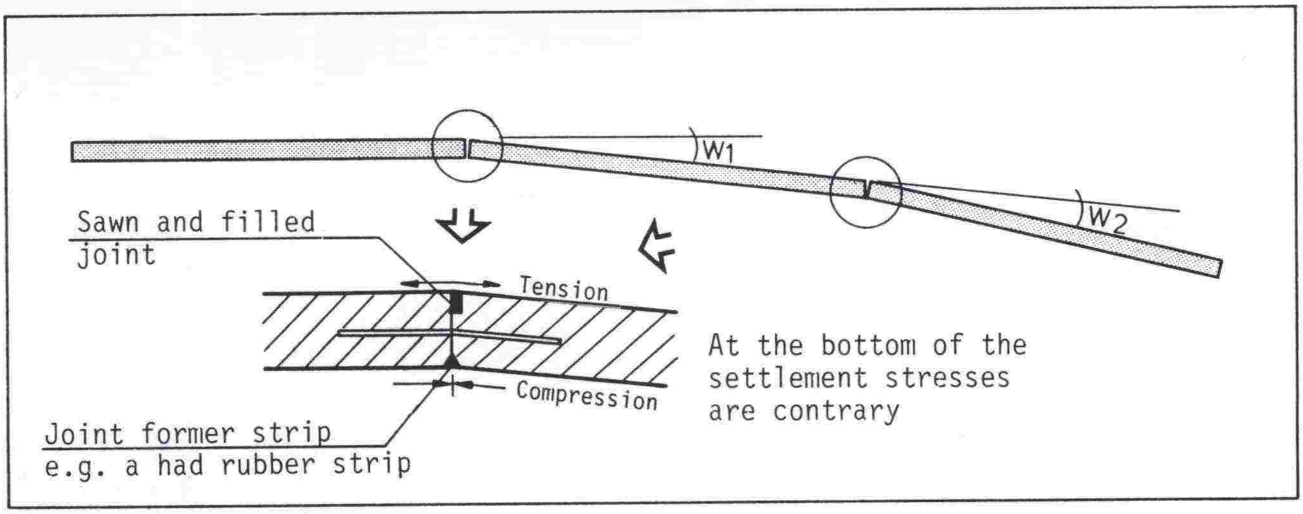


FIGURE B5-14. Stresses and structure of joints on compressible soils

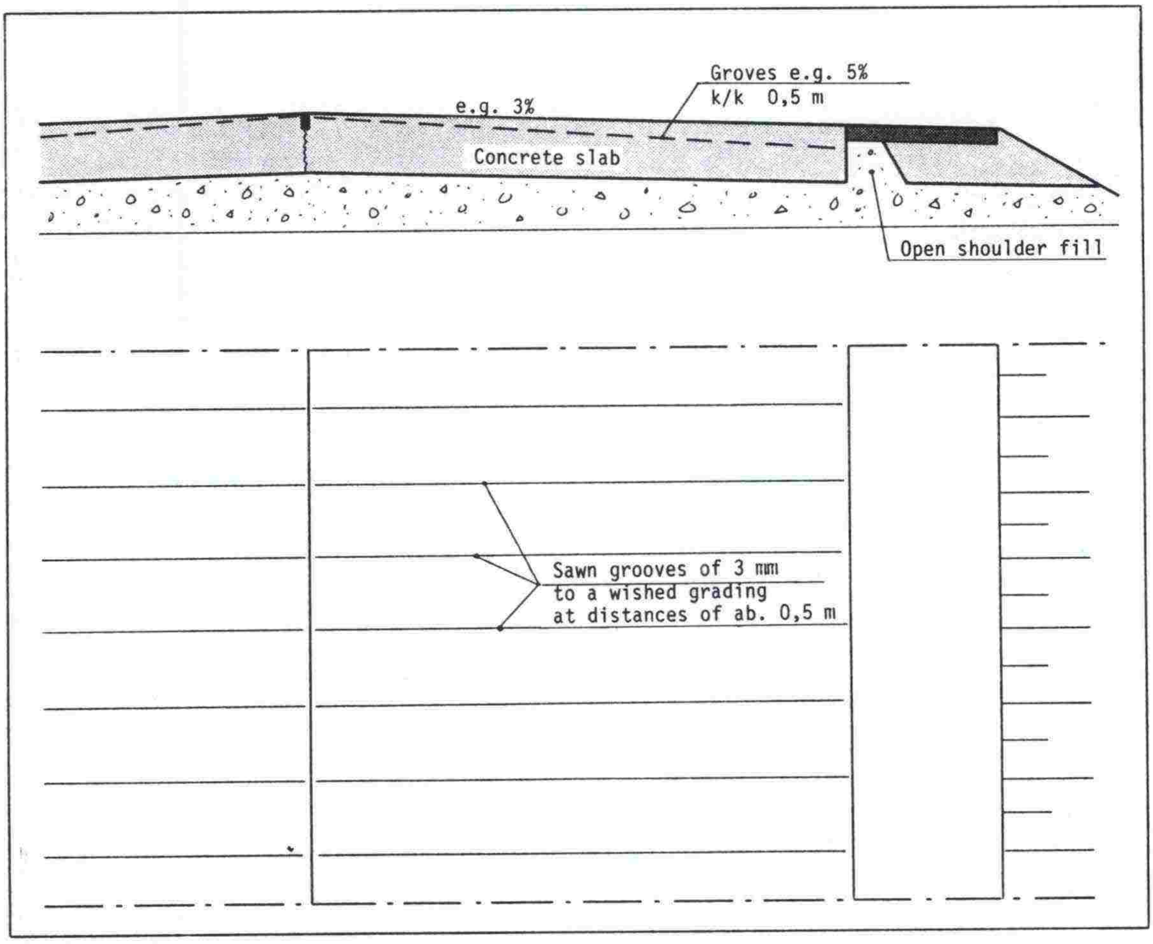


FIGURE B5-15. Increase in the transverse slope by means of grooves

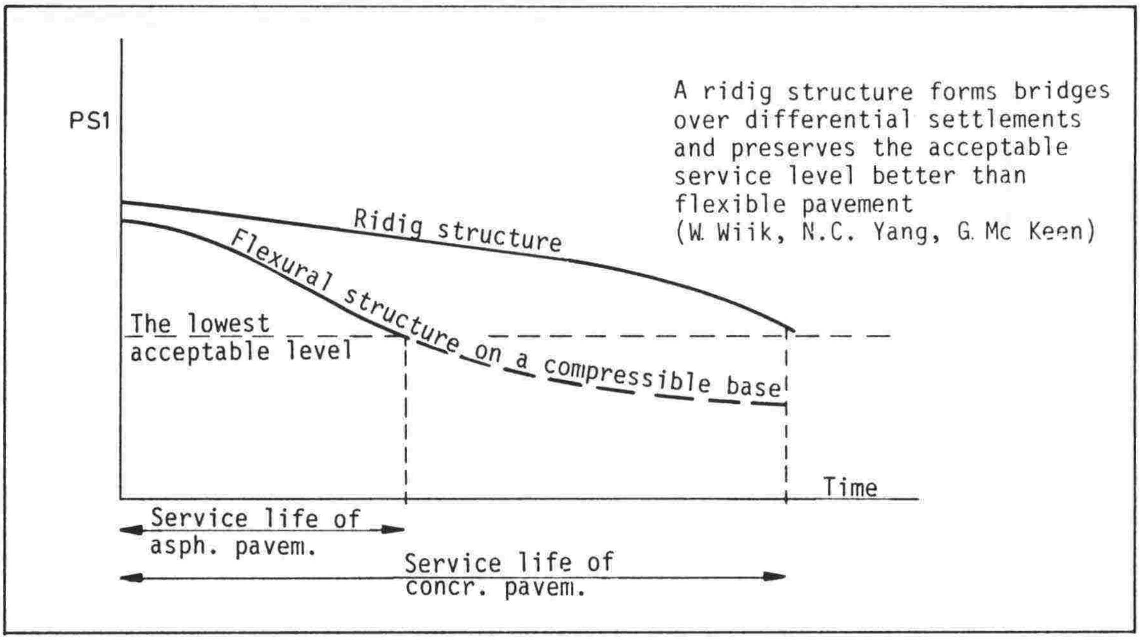


FIGURE B5-16. Share of settlements in the decrease in the service level

B 533 Settlements and concrete pavement in different countries

Concrete pavement is generally regarded as a pavement for incompressible soils. Concrete pavements are avoided on soils sensitive to differential settlements in many countries. In an international survey made in 1982 only Switzerland and Holland showed any design activity regarding concrete pavements on compressible soils. It is usual that asphalt pavements are chosen for compressible soils. Continuously reinforced concrete slabs are recommended in England. High embankments of main roads are first paved with asphalt in Austria. After some years of compaction the final concrete pavement is built. The thickness of the concrete slab is increased on weak and compressible soils in Holland. /6, 7/ Irregularity on roads and especially the risk for breaking the slab are regarded as uncertainty factors there. Concrete pavements are often built on very heavily trafficked roads and this may lead to a quick widening of even minor damage and cause repair need.

This evasive or passive attitude is possible in many countries, because there are few areas with weak and compressible soils and thus the policy of undepressed roads is not technically nor economically overstraining.

Switzerland has an experience of more than 50 years in concrete pavements on weak and compressible soils, Figure B5-17./9,10/ About twenty significant projects to build conrete pavements on weak soils have been completed during the decades. The thickness of peat is usually 2 - 8 m. Peat has often been left as lenses between other soil layers. In these cases concrete pavements have succeeded well, consolidation of no more than 20 cm has taken place without needing considerable repairs. Early projects have encouraged to go on and thus valuable information has been gathered. The latest significant project is the 7 km long motorway section N 3, Flums -Sargans. The road was designed as a sandwich structure on peat soils, the thickness of which was 9 m at the most (compare Figure B5-7). The reinforced slabs are 21 cm thick and 6,25 m long and they are equipped with dowel bars. The condition and settlements of the road have continuously been followed up, Figure B5-18, and the results have been reported in several international congresses. One greater settlement has taken place on the road section (about 45 cm for a distance of 80 m) but inspite of this the service level of the road is still reported to be good (3,8/5,0) and the road is used as a reserve runway for military plains. The pavement photographed in autumn 1987 is shown in Figures B5-19 and B5-20./18/

Incompressible subgrade soil is required for concrete pavements in France. The pavements are cast on a cement-treated layer without dowel bars. Consolidation has been proved to reflect joint damage into the road surface. This kind of a damage took place on Main Road A 25 in northern France (Figure B5-21). It is a question of a high embankment built of slag where rather sudden differential but minor settlement has occurred and this has led to faulting soon after the construction. The road has succesfully been repaired by grinding and by surface treatment.

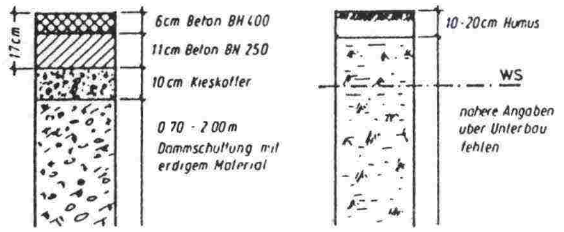
Holland has reported consolidation of as much as 35 cm for a distance of 30 m without pavement cracks. Settling concrete pavements are mainly designed only for inferior roads.

The concrete road Ylikylä-Parainen built in 1958-59 in Finland was completed on a very difficult terrain in terms of consolidation and practically without any subgrade strengthening measures. The 12,5 km long road crosses several compressible coastal soils on relatively high embankments without any subgrade strengthening. Slabs were bound to be replaced at piled culverts due to steep post construction differential settlements. Short sections were paved with asphalt for the same reason. Otherwise, the concrete pavement has kept its service level well inspite of consolidation and this can still be seen, Figure B5-22, although the entire road has been overlaid with thin asphalt in 1984.

The test road Palojärvi - Olkkala /16, 17/ was built in 1973 on an embankment of 1 - 2 m nearly entirely on deep compressible soils supported by dry crust clay. The soil is formed by layers of thick varved clay. Unexpectedly great post construction consolidation started to occur on the road section. Consolidation of as much as 40 - 50 cm occurred on the road in four years, Figure B5-23. The settlements were especially large on full-depth asphalt and concrete sections. A follow-up report /16/ of the test road states that consolidation on concrete sections does not considerably inconvenience driving comfort, because they are relatively gentle and wide.

Kantonsstrasse Quartino - Reazzino

Kanton Tessin Baujahr 1936



Neuaufbau 1:10 Bestehend 1:25

Plattenlänge: 10.00 m

Plattenbreiten: 2 x 3.25 m Fahrbahnbreite: 6.50 m

Abstand der Dilatationsfugen: 40.00 m

Bewehrung: *Armierung 3.20 kg/m²* *Netze* —

Setzungsverhalten: *lokale, bis 6 cm*

Plattenhebungen: *1 Stelle bei Durchlass (1951)*

Rissentwiklung: *total 3 Felder nach 36 Jahren = 0.8 %*

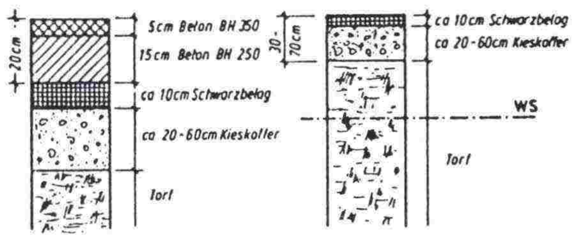
Schwarz - Aufschiftung: *3 Felder (1961)*

Unterhaltskosten nach Ablauf der Garantiezeit:

1941 - 1951 0.28 Fr/m²; 1952 - 1960 0.37 Fr/m²;
1961 - 1966 0.35 Fr/m²; 1967 - 1972 0.47 Fr/m².
≙ Fr 1.47/m² für 36 Jahre

Kantonsstrasse Freienbach-Pfäffikon

Kanton Schwyz Baujahr 1959



Neuaufbau 1:10 Bestehend 1:50

Plattenlänge: 7.50 m

Plattenbreiten: 2 x 4.50 m Fahrbahnbreite: 9.00 m

Abstand der Dilatationsfugen: 37.50 m

Bewehrung: *Armierung 1.06 kg/m²* *Netze 1.95 kg/m²*

Setzungsverhalten: *bis 1960, bergseitiger Strassenrand bis max. 6 cm*

Plattenhebungen: *keine*

Rissentwicklung: *bis 1972 4 Risse in 4 Platten von 162 = 2.5 %*

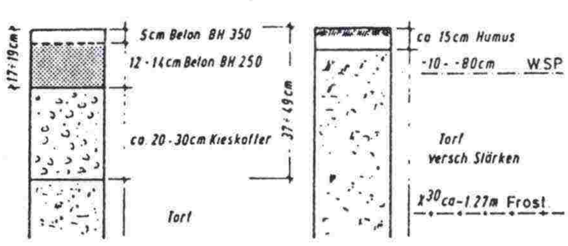
Abbruch: ———

Unterhaltskosten nach Ablauf der Garantiezeit:

1962 - 1972: total Unterhalt 1'305.-Fr. für 8'670 m²
1.5 Rp/m² und Jahr. ≙ 15 Rp/m² in 10 Jahren

Staatsstrasse Höri - Riet. ZH

Baujahr 1951



Neuaufbau 1:10 Bestehend 1:20

Plattenlänge: 7.80 m

Plattenbreiten 2 x 3.00 m Fahrbahnbreite 6.00 m

Abstand der Dilatationsfugen: 31.20 m

Bewehrung: *Armierung 1.4 kg/m²* *Netze 1.5 kg/m²*

Setzungsverhalten *Vertikalbewegung ± 2.5 cm/Jahr durch Frost*

Plattenhebungen *Keine*

Rissentwicklung *total 268 Felder, bis 1971 3 Felder*

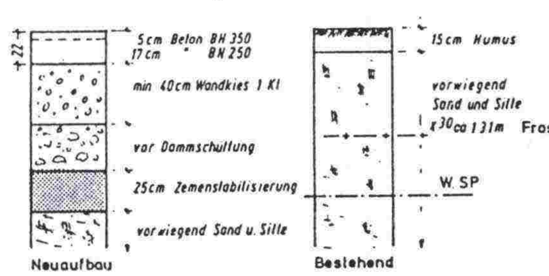
Abruch wegen neuer Kreuzung: *1969 15 Felder*
1970 36

Unterhaltskosten nach Ablauf der Garantiezeit:

1951 - 1956 0 Rp/m²; 1956 - 1961 5 Rp/m²
Total Unterhalt: 1250.- Fr. für 5875 m²
≙ Fr. -.21/m² für 10 Jahre

N3 Flums - Heiligkreuz - Mels

Baujahr 1970



Neuaufbau Bestehend

Plattenlänge *7.50 und 6.25 m*

Plattenbreiten *4 x 4.00 m* Fahrbahnbreite *2 x 8.00 m*

Abstand der Dilatationsfugen *Keine D-Fugen*

Bewehrung *Armierung 1.08 kg/m²* *Netze 1.5 kg/m²*

Setzungsverhalten *10.11.70 - 1.10.71 0.3 ÷ 22 cm Setzungen*

Plattenhebungen *Keine*

Rissentwicklung *Keine*

Schwarz - Aufschiftung teilweise —

" " total —

Unterhaltskosten nach Ablauf der Garantie:

FIGURE B5-17. Examples of Swiss concrete pavements on compressible soils /19/

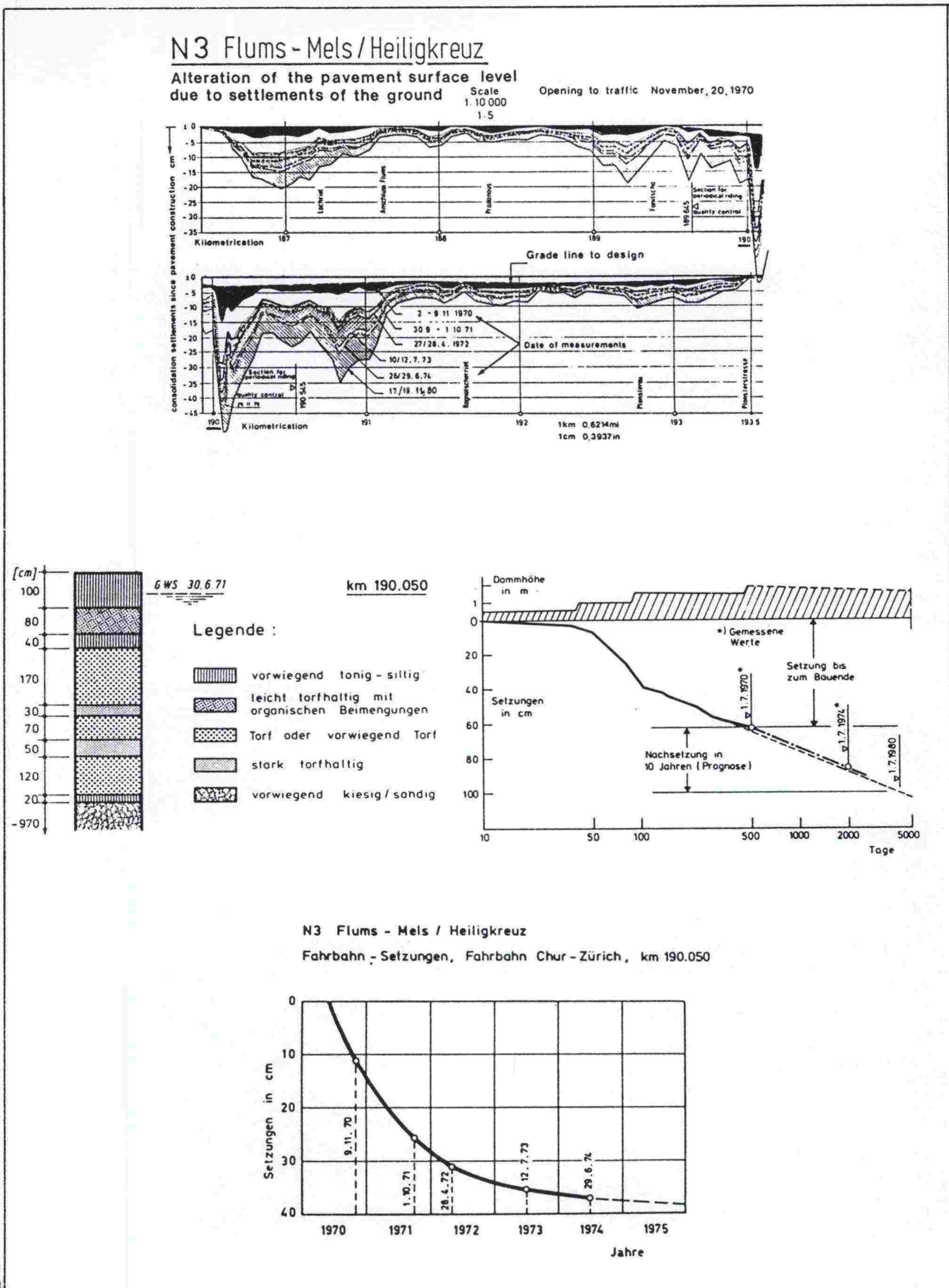


FIGURE B5-18. Settlement measurements of concrete road N3 Flums-Sargans /9/

FIGURE B5-19. N3 Flums-Sargans, Switzerland, settlement of the concrete pavement (45 cm), photographed in 1987

FIGURE B5-20. N3 Flums-Sargans, a general view of the settled embankment in Switzerland. The concrete pavement in an excellent condition in 1987

FIGURE B5-21. A concrete pavement in France damaged due to compaction of the embankment; rehabilitated by grinding and surface treatment (photo JR/87)

FIGURE B5-22. Consolidation settlements on Paraistentie (concrete pavement 1959, asphalt treatment 1984), Finland

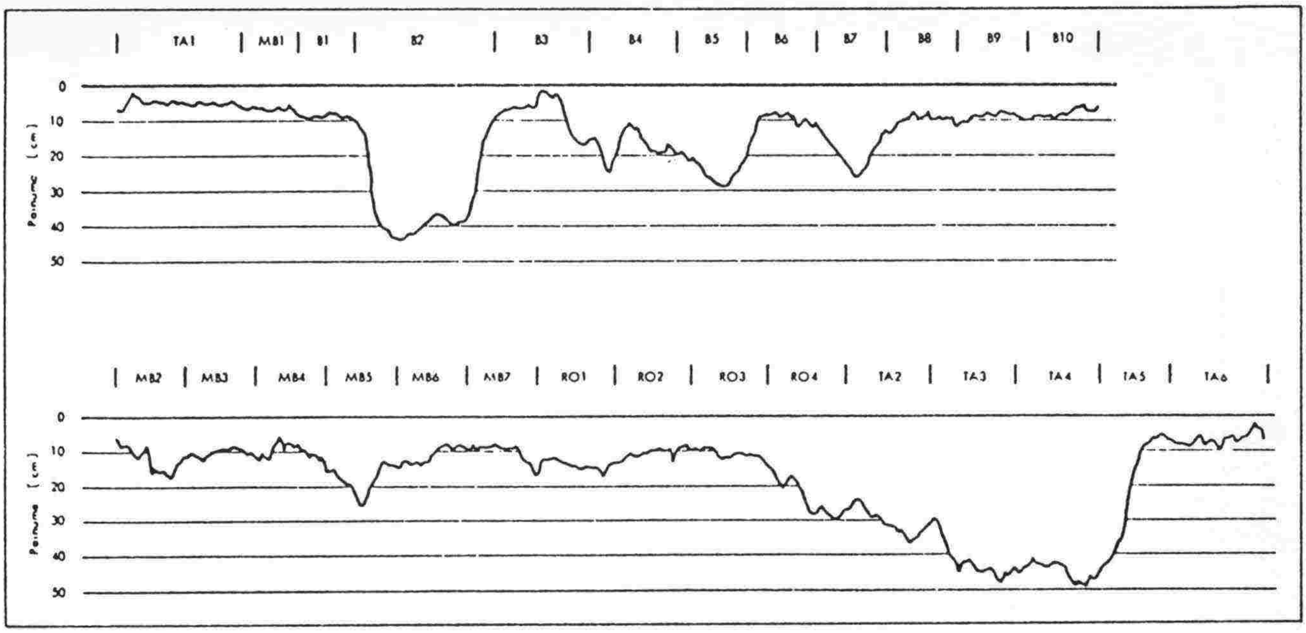


FIGURE B5-23. Consolidation of the Palojärvi-Olkkala test road from autumn 1973 to autumn 1977 /16, 17/

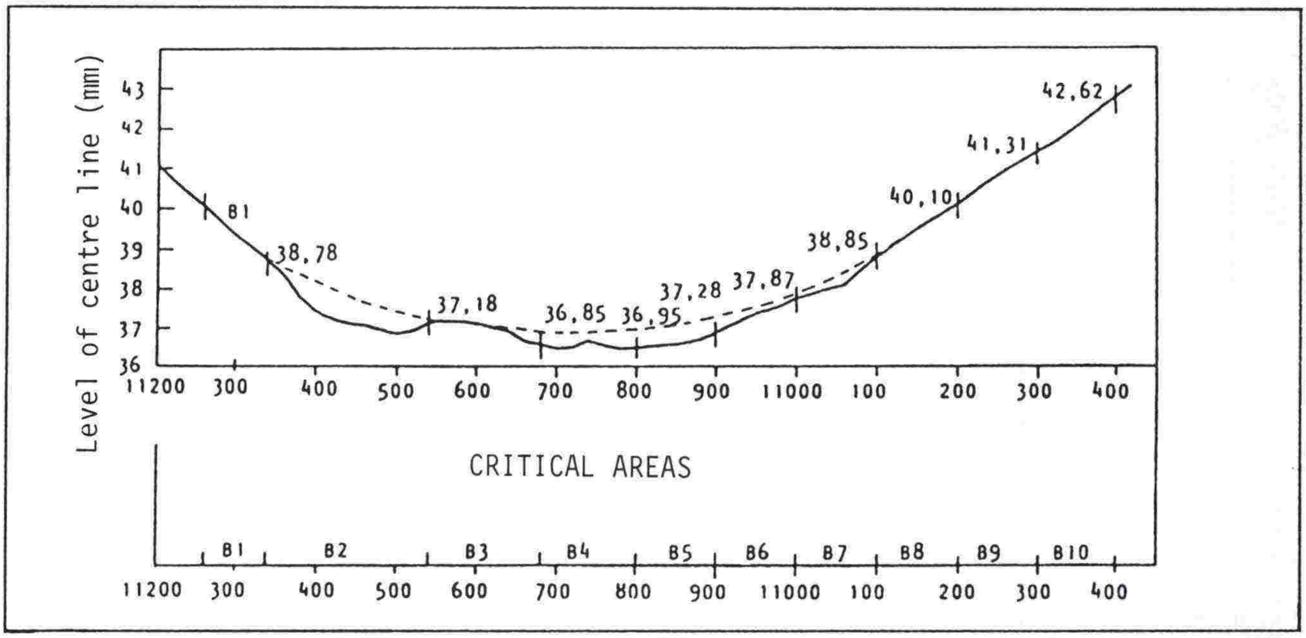


FIGURE B5-24. Level of the centre line of the Palojärvi-Olkkala test road 1973 and 1984 (when the settlements were evened out with asphalt) /16, 17/

According to the report the preservation of the driving comfort results from the fact that "the rigid pavement evens out bumps caused by irregular consolidation of the soil; these bumps have reduced driving comfort especially on full-depth asphalt sections". The test road - except for the concrete sections - was levelled and repaved in 1978. Later it has been necessary to level the concrete sections and pave them with asphalt, because the concrete pavement lost its crossfall and ponded with the advancement of the consolidation, Figure B5-24, (consolidations of 80 cm were measured for a distance of 100 m). Also blow-ups, especially in longitudinal joints, occurred when the slabs moved in relation to each other.

B 534 Key factors of the design

When designing concrete pavements for soils sensitive to differential settlements the demands for the design are to be found in the new Finnish design instructions of concrete pavements /14/. The essential design values are as follows:

- the total settlement of not more than 250 mm during the service life of the road
- the change in the gradient (longitudinal slope fall) 5 o/oo at the most during the service life
- the change in the crossfall 10 o/oo at the most during the service life
- consolidation speed 30 mm/year at the most in the beginning of the service life
- the safety factor of the embankment against slide 1,7...1,8

These design values are necessary in order to reach a controlled design practice. As to numerical values, however, the above specifications are very cautious. The magnitude of the total settlement and the changes in the gradient and crossfall could be considerably greater without compromising the driving comfort. The only important point in the crossfall change is preserving the good flow conditions of water. These design values can be understood as guiding lines and they can differ projectwise if the causes for deviation can be justified.

When a situation as described in the design values cannot be reached by adjustment of the horizontal and vertical alignment in the terrain conditions of the design project, the necessary subgrade strengthening measures are to be designed to increase the stability and to restrict the total settlement and differential settlements. It can be assumed that this procedure leads to a result where the settlement does not shorten the effective service life of the concrete pavement nor decrease its service level.

In addition to the subgrade strengthening measures it is important that the entire pavement structure is designed as rigid as possible. Its thickness should also be as uniform as possible. A cement-treated layer on the lower part of the subbase course can be recommended.

Damage risks caused by consolidation on the slab itself can also be prevented:

- short slabs are used (4,5...6,0 m)
- dowel bars are used (abroad often also reinforced slabs)
- compressible joint formers are placed on the lower surface of the slab
- broad joints are sawn (more than 10 mm, also longitudinal joints) and joint sealant is used

This design principle of the slab is equal to the normal Finnish design procedure; thus the risk for consolidation does not essentially influence the design of the slab.

To be sure that no detrimental consolidation occurs on the concrete pavement, an asphalt layer can be designed as a base for the concrete pavement and the road can be opened to traffic as asphalt-paved and the consolidation situation can be ensured by measurements before paving with concrete.

If, inspite of everything, consolidation occurs on the concrete pavement during the course of time, repair actions won't be taken until the consolidation is detrimental to the traffic. The final repair is undertaken by lifting or replacing the settled slabs. If consolidation is still going on, the settling section is repaired by paving it temporarily with asphalt.

B 54 SUMMARY

Irregularities on the road surface develop for many reasons: finish compaction of the embankment, frost action, fatigue of the pavement structure, abrasion of the pavement or consolidation of the soil. Settlements are always a risk for the service level and the durability of the structure. Most of these risks can be controlled by structural solutions and careful construction. Risks caused by the consolidation of the soil are controlled by defining the allowed consolidation and differential settlements for each road category and by taking the necessary subgrade strengthening measures. The pavement type does not affect the magnitude or speed of the consolidation, but the concrete pavement is capable of evening out local differential settlements better. Structurally, the asphalt pavement withstands greater differential settlements than the concrete pavement without breaking, but both pavement types endure more than can generally be allowed as far as the service level of traffic is concerned. The service life of concrete pavements being long more differential settlements will occur on them than on asphalt pavements. If settlement repairs are allowed to be made as asphalt overlays, the geotechnical design of the pavement types need not be different, but in other cases a longer service life of concrete pavement must be taken into account as better subgrade strengthening methods.

Finland has good experiences of designing embankments on weak and compressible soils. This experience can be utilized also when designing concrete pavements on soils sensitive to differential settlements; the fact that concrete pavements are suitable for evening out differential settlements can thus be utilized as a good evenness of the road.

CHAPTER B5 CONCRETE ROADS ON WEAK AND COMPRESSIBLE SOILS

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CHAPTER C

CONCLUSIONS AND RECOMMENDATIONS

CHAPTER C

CONCLUSIONS AND RECOMMENDATIONS

CONTENTS Page

CHAPTER C
CONCLUSIONS AND RECOMMENDATIONS

The applicability of cement-bound structures in the Finnish traffic, soil and climate conditions has been studied in this survey. The material dealt with has been outlined in such a way that the knowledge of the possibilities and restrictions of the cement-treated and concrete pavements could diffuse to wider and wider circles. In the summary of each chapter (A, B1...B5) a general view of the issue and research and development needs has been presented. Conclusions should be drawn by the reader himself. The author of the report will, however, give his own viewpoints of the conclusions and recommendation of the report in the following.

C 1 CONCLUSIONS

C 11 Conclusions of the usage of cement-treated materials when improving the load-bearing capacity in the road and city street construction

On the basis of the summary of Chapter A 11 the following conclusions can be drawn:

- Good experiences have been obtained of the usage of cement-treated materials as a subbase and base course of asphalt-paved roads and city streets. It can be assumed from the wide international interest shown that the usage will considerably increase all over the world in the 1990s.

- Reflection cracking and other risks typical of cement-treated materials can now be controlled. This has meant stricter and stricter demands for the aggregate and for the proportioning and construction work of cement-treated layers.

- Mixed-in-plant and mix-in-place methods should be used and developed side by side and on the basis of the experience the most suitable objects of use for each method in the Finnish conditions should be decided. Good experiences of mix-in-place projects completed with efficient special equipment have lately been obtained in Finland. It is generally required abroad that cement-treated mixes on the base course should be made by the mixed-in-plant method.

- Also negative experiences have been received of cement-treated materials in cold regions - in Scandinavia and North America and thus their use has been little. A defective frost protection has often been the cause for damage. On the basis of the material dealt with in this report, a cement-treated layer which is designed for the subbase or base course can be considered suitable for the Finnish conditions. However, the following points should be emphasized:

  - adequate drainage of the structure
  - sufficient frost protection
  - a sufficient load-bearing capacity of the subbase
  - good frost resistance of the cement-treated layer
  - a sufficient thickness of the asphalt pavement to prevent crack reflection when the pavement is built directly on the cement-treated layer.

- The competitiveness of the cement-treated pavement provides that the good load-bearing capacity and long service life of the pavement are taken into account in the design and that the use of cement-treated materials influence also material choices of other structural courses. Thus the use of cement-treated pavements proves to be the most profitable way to increase the load-bearing capacity of heavily trafficked roads and city streets and to decrease their deformation.

C 12 Conclusions of the technical and economical qualifications of concrete pavements in Finland

Concrete pavements have been dealt with in Chapters B1...B5. The summaries of these chapters lead to the following conclusions:

- (Chapter B 1) Construction methods and equipment have developed very quickly and contributed to a more flexible use and to a more efficient production of concrete pavements. The characteristcs of a modern slip-form machinery are:
    - an automatic installation of dowel bars
    - a bending beam
    - a possibility to reinforce the slabs
    - a possibility to steplessly change the width and thickness
    - a levelling controlled by a laser beam

The new equipment contributes to a greater flexibility in the concrete pavement construction e.g. on one-lane roads, intersections, ramps etc.

- (Chapter B 1) The success of the construction work has a decisive significance to the service level and durability of the concrete pavement. The succes calls for good equipment and materials and a highly-skilled working team. There have always been bad defiencies in these factors when concrete pavements have been built in Finland in the past few years. International contracting would be an appropriate way to guarantee the high quality of the work.

- (Chapter B 2) In the 1980s good methods and equipment have been developed to repair and rehabilitate concrete pavements. There are also several methods to repair ruts. Of these diamond grinding seems to be the most suitable repair method. However, there is not enough knowledge of the repair and rehabilitation methods in Finland. Hence, it is necessary to make out a manual of our own and to arrange controlled repair projects.

  Joint sealants should be changed at intervals of about 5 years and ruts should be repaired by grinding on the most heavily trafficked roads. Other repair needs depend on how well the construction work has succeeded.

  The service life of the concrete pavement can be 35 - 40 years, if one significant rehabilitation project is provided for. The old pavement is left as a base of the new pavement either as such or as crushed at the end of the service life.

- (Chapter B 2) The traffic management related to repair works have become much easier thanks to quickly hardening concrete mixes. If needed, the repair work can be performed so that the repaired area is opened to traffic only after a few hours after the casting.

- (Chapter B 3) The wear resistance of aggregates is the main factor in the studded tyre wear of pavements. As to the wear of concrete pavements the strength of concrete is another decisive factor. If the strength of the concrete pavement ranges between K25...K35, there is no essential difference between the rutting of concrete and asphalt. But when the strength of conrete reaches a level of K60...K70, the durability of the concrete pavement will be 2,5...3,5 times higher than that of the asphalt pavement built of the same aggregates. A test proved that the wear resistance of concrete was twice as much as that of the rubberized bitumen. The advantage of concrete proves to be so great that the best aggregates should be cast as concrete pavements where repeated repairs of rutting asphalt have become a problem. The use of concrete and special aggregates extends the intervals of rut repairs many times over. Thus the need for rut repairs on concrete pavements becomes apparent only on the most heavily trafficked roads and the repairs can be performed by grinding at a moderate cost.

- (Chapter B4) The frost protection of the road is carried out to even out the differences of the spring bearing capacity and frost heave. The category and speed level of the road determines how strict requirements for load-bearing and frost heave differences are set. The concrete pavement evens out differences in the load bearing capacity well and no frost protection layers would be needed for this purpose. But to preserve the regularity and service level of the concrete pavement differential frost heave is necessarily to be evened out by pavement layers corresponding to the procedure on asphalt-paved roads. Compromising in this principle has led to a reduced regularity and also to other damage, among others on motorways in cold regions in the United States in the course of time.

  On the other hand, thicker frost protection layers should not be demanded of concrete pavements than of asphalt pavements, because the concrete pavement (especially the pavement built in Finland: short slabs, equipped with dowel bars) can structurally withstand deformation due e.g. to frost heave when the design has been made according to the Finnish design practice.

- (Chapter B 5) When the embankment is built on soft soil deposits consolidation and differential settlements occur in the course of time and they reduce evenness and crossfall. Due to the long service life the concrete pavement consolidates more and this can require repairs to improve the service level of the road. If these repairs are made with asphalt the use of the two pavement types need not be different on weak and compressible soils. However, it is generally assumed that the concrete pavement should preserve its service level without repairs during its entire service life. This calls for additional subgrade strengthening measures to even out differential settlements and to accelerate consolidation when the construction takes place on weak and compressible soils.

  As to main roads the geotechnical design demands are set by the road category and no extra costs are caused by the use of concrete pavements. As to secondary roads subgrade strengthening measures cause additional costs on concrete roads if later consolidation repairs are to be prevented.

- The most suitable concrete pavement for the Finnish traffic, climate and soil conditions is an unreinforced pavement with rigth-angled short slabs and dowel bars. An asphalt-paved roller compacted or normal concrete pavement should be investigated as a new solution. This kind of an alternative is included at least in the Spanish, English and Canadian road standards. It could be the most profitable solution on all heavily trafficked roads or on roads sensitive to differential settlements in Finland. The use of an asphalt overlay reduces the thickness of the concrete pavement, decreases demands for evenness and, when using rcc, accelerates the opening to traffic; at the same time all the structural advantages of the concrete pavement are preserved.

- On the basis of the material dealt with in this report it can be stated that there are good technical qualifications for the use of concrete pavements also in the Finnish conditions. Cold climate sets special requirements for the road construction but we are used to solve them here and the concrete pavement does not considerably change the solutions. The most natural area of use for concrete pavements both technically and economically would be main roads and streets or other heavily trafficked roads and streets in Finland. When these kinds of roads are built the most important advantages of the concrete pavement are its load-bearing capacity, non-deformation, wear resistance and long service life.

- As to the price the concrete pavement is unable to compete directly with unbound pavement layers. On the other hand, on bound layers of main roads and streets the concrete pavement is competitive already in its initial price. And thanks to smaller maintenance costs the competitiveness of the concrete pavement will be improved even further when the long-term costs are included in calculations.

C 13 Unbound, flexible or rigid pavement structure?

The designer or the decision maker should not regard the choice of the pavement structure as self-evident or as a simple either/or task, it should be a result of wide comparisons and judgement. Factors relating to cement-bound structures - and only certain specially chosen issues of them - have been dealt with in this report. As there is no possibility to a more thorough comparison of different structural types in the Finnish conditions in this connection, a newly published Central European recommendation of the comparison performance is shown in Tables C-1 and C-2. Table C-1 studies technical and Table C-2 economical and industrial viewpoints. The approach is theorethical and rough and it may not always be directly adapted to the Finnish situation, but the study deals with topical issues and encourages Finnish designers and decision makers to more and more versatile investigations when pavement and structure types are chosen.

Many factors have favoured the use of unbound road structures in Finland, e.g.:

- the relatively good availability of ridge aggregates
- healthy, good aggregates
- the thick, non-frost-susceptible structure demanded by the climate
- no strict environmental requirements (no spoil prohibitions, no recycling compulsion for materials etc.)

- no industrial waste mountains which should be 'buried' into the road structure
- moderate traffic flow and stress

The use of bound structures is increasing in Finland as also in other developed countries along with

- the increase in heavy traffic
- the increase in axle and unit weights
- the increasing need for strengthening the existing road network
- the stricter and stricter environmental requirements for the use of material

Using cement-bound structures is not always the only - or the right - way to build a bound pavement structure. But - as stated above - there are many applications where cement-bound structures could prove to be advantageous also in our circumstances. The Finnish comparison principles and design and construction practice should be thought out in connection with our new concrete road projects.

TABLE C-1. A FLEXIBLE, RIGID OR COMPOSITE STRUCTURE; favourable (+) and unfavourable (-) factors in a technical comparision of the different structure types

TYPES OF PAVEMENT / SIGNIFICANT TECHNICAL PARAMETERS		FLEXIBLE			SEMI-RIGID		COMPOSITE	RIGID	COMMENTS
		Untreated gravel	Sub-base : untreated gravel Road base : gravel treated with bituminous binders	All gravel treated with bituminous binders	Sub-base : untreated gravel Road base : gravel treated with hydraulic binders	All gravel treated with hydraulic binders *	Sub-base : gravel treated with hydraulic binders Road base : gravel treated with bituminous binders	Cement concrete (sub-base treated for heavy traffic)	
TRAFFIC Initial number of heavy vehicles with a pay-load greater than 50 kN (per day in each direction) Legal limit for axle loads									- Pavements composed with hydraulic binders are very sensitive to overloads which have not been provided for in the thickness design calculations.
130kN	100kN								- Conversely, a slight extra thickness makes it possible to account for overloads when these are expected.
< 40	< 100	++	-	--	+	-	--	+	
40 - 100	100 - 300	+	+	--	-	+	--	-	- Low initial traffic and a high rate of growth (> 10 %) are factors which favour the use of untreated gravel, together with stage construction.
100 - 700	300 - 2000	-	-	+	--	++	+	-	
> 700	> 2000	--	--	++	--	-	++	++	
SUBGRADE : bearing capacity Very high (CBR > 20 or ME_{II} > 150 MP_a)		++	++	-	+	-	+	-	- High-quality subgrade soils are favourable to the use of flexible pavements.
Fair (20 > CBR > 6)		-	+	+	-	+	+	+	- It is generally advised not to construct rigid pavements on compressible soils.
Poor (CBR < 6)		--	-	-	--	++	+	++	
Differential settlements expected		++	+	-	-	--	--	--	- ME_{II} : modulus at the second cycle of the plate-bearing test.
CLIMATE Very high temperatures (risk of rutting)		-	--	--	++	++	--	++	- The risk of rutting is greater on upwards grades.
Severe frost		--	-	+	+	++	+	++	- Pavements in which loads are well distributed are less sensitive to a loss of bearing capacity during a thaw.
Heavy rainfall		-	+	++	+	++	++	++	
Presence of de-icing salts		-	-	-	-	-	-	+	- De-icing results in maintaining a presence of water on the pavement, which is detrimental to the behaviour of bituminous surfacings.

* Gravel treated with cement or some other hydraulic binder.

TABLE C-2. A FLEXIBLE, RIGID OR COMPOSITE STRUCTURE; favourable (+) and unfavourable (-) factors in an economic and industrial comparision

TYPES OF PAVEMENT / SIGNIFICANT ECONOMIC AND INDUSTRIAL PARAMETERS	FLEXIBLE			SEMI-RIGID		COMPOSITE	RIGID	COMMENTS
	Untreated gravel	Sub-base : untreated gravel Road base : gravel treated with bituminous binders	All gravel treated with bituminous binders	Sub-base : untreated gravel Road base : gravel treated with hydraulic binders	All gravel treated with hydraulic binders	Sub-base : gravel treated with hydraulic binders Road base : gravel treated with bituminous binders	Cement concrete (sub-base treated for heavy traffic)	
Limited immediately available funds and/or high discount rate	++	+	+	-	-	-	--	- Bituminous pavements which tolerate a time-staged strategy have a relative advantage. - The insufficient size of work sections is relatively unfavourable to cement concrete.
Cost of hindrance to traffic due to maintenance works	-	+	+	-	-	+	+	- This point should be examined in detail with a view to traffic, maintenance sequences, and the value of users' time.
Existence of unused sources of hydraulic binders (slags, fly-ash, pozzolans or cement)	-	-	--	+	++	+	++	- Provided there is a willingness to implement an adapted industrial strategy. - Concrete can be made using inexpensive cement with a low clinker content.
Relative scarcity of road bitumen	-	-	--	+	+	-	++	- This problem is connected with that of the balance of trade if bitumen is an imported product.
Scarcity of high-quality aggregates	--	-	-	+	+	-	++	- This parameter favours techniques which require less aggregates and tolerate local or lower quality aggregates.
Concern for energy savings (in investments + maintenance)	-	-	--	-	+	-	+	- On the assumption that bitumen is a potentially energetic product (9,500 thermal units/ton).
Risk of inadequate quality of construction	(1)	(1)	(1)	-	-	(1)	--	- This parameter expresses the sensitivity of the techniques to the quality of construction. Concrete presupposes greater technical skills and thus requires a favourable quality context

(1) All techniques require a high quality of execution. The techniques for which a defect in execution (thickness, quality of materials) may be irreversible are marked - or - -

C 2
RECOMMENDATIONS

The author of the report wishes to present the following recommendations for the planning of further actions and for the utilization of the cement qualifications.

- Matters related to the use of cement should be part of the pavement policy as a whole, Figure C-1. It should be noted that research, training and specification actions are being taken all the time in respect of structures bound with bitumen. After having these functions properly started also regarding cement-bound materials they should be merged as a normal part of the road construction practice.

- Thorough training programmes for cement-bound structures should be carried out for different key groups and students at universities and technical highschools.

- Possibilities to accomplish a concrete road programme are to be studied as a co-operation of the different parties. The minimum demand of the first phase would be annual construction of concrete pavements of about 8 - 10 km for example during 5 years. In addition to new motorway and main road sections also present asphalt-paved road sections of heavily-trafficked urban roads and streets could be chosen for the concrete road programme.

- The quality of concrete pavements will be ensured by using foreign equipment and craftsmanship in the initial phase.

- Construction and maintenance specifications for concrete pavements should be drawn up in co-operation with the parties.

- Cement-treated structures (= composite structures) are to be investigated in connection with the ASTO-project or by special projects.

- Specifications for the use and design of bound base courses are to be made out in the Road and Waterways Administration.

- Research and development activities are to be continued. Development needs are described in the summaries of the various chapters of the report. In addition to the above attention may be paid at least to the continuation of morain investigations, testing of the two-layer technique in the construction of concrete pavements and to the study of asphalt-overlaid concrete pavements.

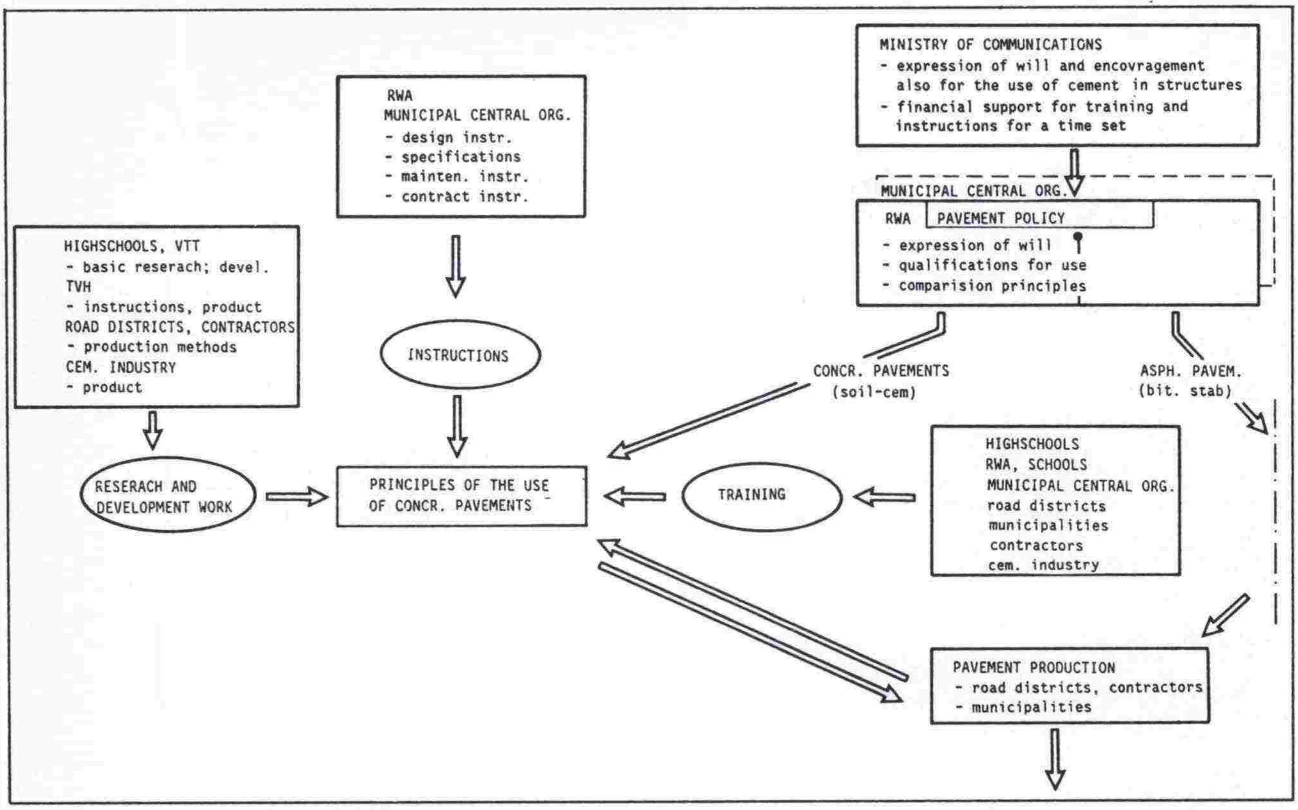

FIGURE C-1. Associating concrete and cement-treated pavements with pavement policy

ABBREVIATIONS

ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials (formerly AASHO)
ACI	American Concrete Institute
ACPA	American Concrete Pavements Association
ASCE	American Society of Civil Engineers
ASTM	American Society for Testing and Materials
ASTO	Asphalt Pavement Research Program in Finland - started 1986
BAST	Bundesanstalt fuer Strassen-Wesen
CBI	Cement och Betonginstitut
C&CA	Cement and Concrete Assosiation
CEMBUREAU	Organisation of European Cement Producers (Paris)
(C) CP	(Cement) Concrete Pavement
-PC(am) (URC)(engl.)	- Plain Concrete Pavement
-PD	'Plain Dowelled'
-RD	'Reinforced, dowelled'
-CRC(P)	'Continuously reinforced'
CRREL	US. ARMY Regions Research and Engineering laboratory
CTGM	Cement Treated Granular Material (Cement treated base)
FHWA	Federal Highway Administration
IRF	International Road Federation
IRRD	International Road Research Data
LCPC	Laboratoire Central des Ponts et Chaussees
NCHRP	National Cooperative Highway Research Program
PANK ry	A Board of Asphalt Paving Specialists in Finland
PCA	Portland Cement Association
PIARC	Permanent International Association of Road Congresses

PSI	Pavements Serviceability Index 4-5 very good 3-4 good 2-3 fair 1-2 poor 0-1 very poor
PSR	Present Serviceability Rating
PTL - NVF	Pohjoismaiden Tieteknillinen Liitto - Nordiska Vägtekniska Förbund - Organisation of Road Speciality in Scandinavian Countries
4-R-program	Resurficing - Repair - Rehabilitation - Reconstruction - Program
RCC (P)	Roller Compacted Concrete Pavement
SC	Soil-Cement
SHRP	Strategic Highway Research Project
State DoT	State Department of Transportation
TRB	Transportation Recearch Board
TRRL	Transport and Road Recearch Laboratory (Crowthorne)
US Army Corps of Engineers	- organisation of engineers (USA)
VTI	Statens Vägtekniska Institutet
VTT	Valtion Teknillinen Tutkimuslaitos (State Technical Recearch Center in Finland)

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